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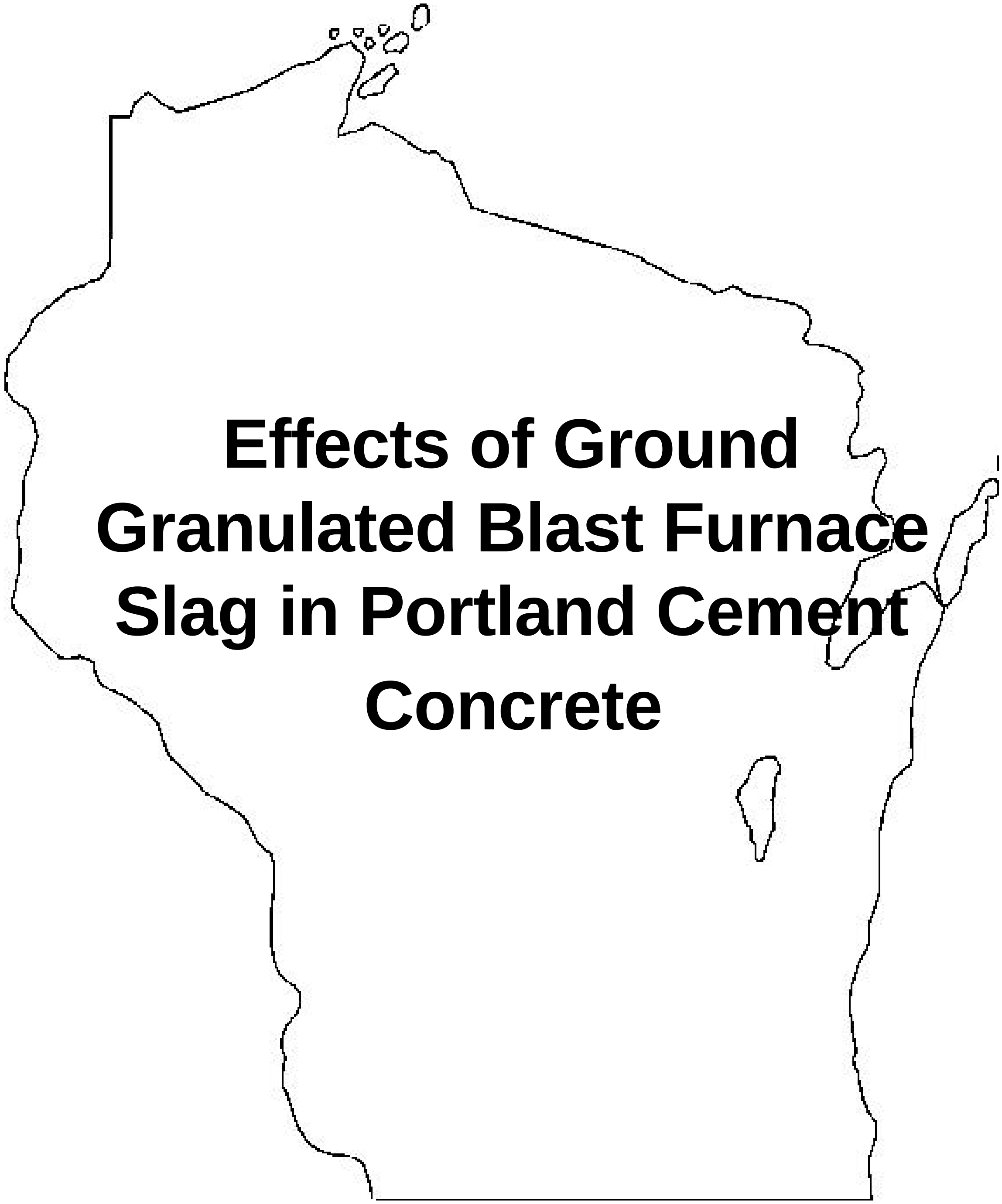

Effects of Ground Granulated Blast Furnace Slag in Portland Cement Concrete

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February 2005

WHRP 05-04

WISCONSIN HIGHWAY RESEARCH PROGRAM #0092-02-14a

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Final Report

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SUBMITTED TO THE WISCONSIN DEPARTMENT OF TRANSPORTATION

February 2005

Acknowledgments

The authors gratefully acknowledge the support of the Wisconsin Highway Research Program for financial support of this project. Accomplishment of the research was achieved through the valued contributions of a team of experts. The cooperation of member companies of the Wisconsin Concrete Pavement Association is appreciated with special acknowledgement to the cement and slag suppliers to the Wisconsin concrete paving market who donated materials to enable this research. The assistance of William Lang of the Structures and Materials Testing Laboratory at the University of Wisconsin-Madison is appreciated.

Disclaimer

This research was funded through the Wisconsin Highway Research Program by the Wisconsin Department of Transportation and the Federal Highway Administration under Project # 0092-02-14a. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Wisconsin Department of Transportation or the Federal Highway Administration at the time of publication.

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Technical Report Documentation Page

1. Report No. 05-04	2. Government Accession No	3. Recipient's Catalog No
4. Title and Subtitle Effects of Ground Granulated Blast Furnace Slag in Portland Cement Concrete		5. Report Date February 2005 6. Performing Organization Code Univ. of Wisconsin-Madison
7. Authors Sippel, C. and Cramer, S.		8. Performing Organization Report No.
9. Performing Organization Name and Address University of Wisconsin-Madison Department of Civil and Environmental Engineering 1415 Engineering Drive, Madison, WI 53706		10. Work Unit No. (TRAIS) 11. Contract or Grant No. WisDOT SPR# 0092-02-14a
12. Sponsoring Agency Name and Address Wisconsin Department of Transportation Division of Transportation Infrastructure Development Research Coordination Section 4802 Sheboygan Ave., Box 7065 Madison, WI 53707-7910		13. Type of Report and Period Covered Final Report, 2001-2005 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

This research examined the impact of cement replacements with Grade 100 ground granulated blast furnace slag (GGBFS) on portland cement concrete performance. GGBFS was used to replace 0%, 30% and 50% of cement in a series of mixes with w/cm = 0.45 where primary variables were coarse aggregate type, cement manufacturer, and curing regime. The primary performance measures were compressive strength development and deicer freeze-thaw scaling resistance. The results show that the amount of time needed to reach 3000 psi traffic opening strength more than doubled from 3 days to 7 days with 30% GGBFS and to 10 days with 50% GGBFS. GGBFS concrete strength becomes comparable to ordinary portland cement concrete after 56 days. Deicer freeze-thaw scaling tended to increase with increasing GGBFS levels and appeared to be tied to the level of carbonation at the surface. Traditional curing methods were less effective with GGBFS concrete in providing a durable surface. In summary, under certain conditions Grade 100 GGBFS can be used successfully in Wisconsin pavements. The seemingly minor variations that result from different mix constituents in OPC appear to be accentuated in GGBFS concrete. A 50% GGBFS cement replacement level usually results in unsatisfactory performance from primarily a scaling perspective. A 30% GGBFS cement replacement level will often be acceptable but the outcome depends on the specific constituents and curing methods used.

17. Key Words Portland cement concrete, ground granulated blast furnace slag, scaling, deicer distress, strength development, durability		18. Distribution Statement No restriction. This document is available to the public through the National Technical Information Service 5285 Port Royal Road Springfield VA 22161	
19. Security Classif.(of this report) Unclassified	19. Security Classif. (of this page) Unclassified	20. No. of Pages	21. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Executive Summary

Project Summary

Ground granulated blast furnace slag (GGBFS), a byproduct from steel production, is being used with increasing frequency as a partial replacement of cement in portland cement concrete. Because it constitutes a beneficial reuse of a by-product material from steel making, federal directives state it must be considered in projects which receive federal funding. While ordinary portland cement concrete (OPC) is a relatively robust material that can be successfully produced under a variety of conditions, the track record with GGBFS concrete is reportedly mixed. In Wisconsin, GGBFS concrete has been used both successfully and unsuccessfully. The reasons for the inconsistency are unknown and it is not even clear if the poor performance can be attributed to the GGBFS. The objectives of this research were to quantify the strength development and durability performances of GGBFS concrete over a range of cement brands, aggregates and curing conditions used in Wisconsin.

Background

With more GGBFS entering the Wisconsin market, it is important to understand the range of its performance limits. GGBFS is known as a latent hydration material, which results in lower early strength development than portland cement concrete. A study done by Lim and Wee showed that with replacements of 50% and 65% Grade 80 GGBFS, strengths were considerably lower than original portland cement concrete (OPC). However, at seven days the strengths had already slightly surpassed those of OPC (Lim and Wee, 2000). The strength development of GGBFS concrete depends on several factors including, GGBFS replacement level, chemical composition, hydraulic reactivity, particle fineness, and curing temperature (Babu and Kumar, 2000).

Along with strength development, the deicer freeze-thaw scaling resistance of concrete is crucial for Wisconsin road construction projects. The slower hydration of GGBFS concrete can be a concern when projects are being completed towards the end of the construction season and the temperature drops. This can only further slow the hydration, making the concrete more susceptible to scaling. Replacement of cement with GGBFS has been known to produce conflicting results with regard to salt scaling resistance (Afrani & Rogers 1994, Stark & Ludwig 1997). Stark and Ludwig found evidence by a majority of authors that GGBFS lowers scaling resistance, but they found some that disagreed. Such ambiguity has led to a blanket standard on the acceptable levels of GGBFS by the Quebec Ministry of Transportation. They limit the level of replacement to 25% based on the sometimes poor performance of higher replacement concretes (Hooten 2000, Afrani & Rogers 1994). ACI Committee 233's stand on the issue is somewhat vague. They have shrugged off the results of laboratory tests and partly base their opinion on a 1967 study by Klieger & Isberner that found minor differences in field applications between concrete with GGBFS and that without. The committee says that research indicates that scaling occurs when both the w/c ratio and the level of replacement are high. Unfortunately, no guidance to what the committee means by "high" is given. Hogan and Meusel in 1981 found little difference between deicer scaling blocks of 100% portland cement and blocks of a 50/50 mixture of cement and GGBFS at a w/c ratio of .53. Both exhibited moderate to severe scaling after 300 cycles. They cast 3 in. by 6 in. by 12 in. blocks and moist cured them for 14 days followed by 14 days of air curing. Water was added to the blocks, and they were subjected to a freeze/thaw regimen at -17.8°C and 21.1°C, respectively. Flaked calcium chloride was added to the ice at the start of each thawing and the block was washed off at the finish of each thawing cycle. ACI Committee 233 bases their stand partly on this study. This may be unjustified since the salt was added after freezing took place and washed clean before being refrozen. This would eliminate the additional osmotic pressures caused by the deicer salt.

While it is known that GGBFS has a detrimental impact on strength development and deicer scaling resistance, it is less clear as how this information should be used for Wisconsin paving projects. Unlike earlier studies, this study aimed to determine GGBFS concrete performance variations over a range of concrete-making materials used in Wisconsin. The research plan sought to quantify the effects on GGBFS concrete of Type I portland cement from four different manufacturers, three GGBFS replacement levels, and two types of coarse aggregate. This research represents the most comprehensive assessment of GGBFS to date.

The research plan consisted of four tasks:

- Task 1: documentation of Midwest GGBFS experiences and a literature survey update,
- Task 2: monitoring of the variability of GGBFS composition,
- Task 3: strength gain and air void development with 4 different brands of Type I portland cement at varying temperatures, and
- Task 4: deicer scaling tests with two different cements and with four different curing methods.

The strength gain and deicer scaling tests used GGBFS as a replacement of portland cement at 0%, 30%, and 50% levels. Thirty mixes were evaluated in the Task 2 regarding strength gain task, and thirteen mixes were evaluated in Task 3 regarding the deicer scaling task. Freeze-thaw testing for the deicer scaling tests followed a modified ASTM C672 procedure where testing was extended to 100 cycles, and the scaled off material from the block surface was weighed every 5 cycles.

Research Process

The methodologies of the four tasks of the research followed accepted procedures with some modifications to enhance the level of meaningful data. **Task 1** established the recent experiences with GGBFS in paving concrete. A survey of neighboring Midwestern state DOT's attempted to find potential commonality of field and performance problems. **Task 2** was targeted at assessing the variability of GGBFS from one producer as delivered to a local ready-mix plant. The tests used to quantify variability were chemical composition of the portland cement and GGBFS (ASTM C114), slag-activity tests (ASTM C989), and particle size distribution. **Task 3** was directed at measurement of air dry shrinkage, hardened air void, and compressive strength test results using the appropriate ASTM standards. Thirty mixes were completed in Task 3. The mix variables were cement brand, curing temperature, GGBFS replacement level, and aggregate type. The purpose of **Task 4** was to quantify the deicer scaling resistance of the GGBFS concrete using a modified ASTM C672 procedure. Task 4 had similar mix variables to Task 3, except that the test concrete utilized only two cement brands. The mix proportions for both Tasks 3 and 4 were based on WisDOT Grade A and Grade A-S designs. A water-cementitious material ratio (w/cm) of 0.45 was used with a target air content of 6% ± ½ % for all mixes. Task 4 tests included slump, plastic air content, unit weight, and deicer scaling resistance according to ASTM C672.

Materials were selected based on their pertinence to Wisconsin concrete paving operations and included the following materials:

1. Type I cement from four manufacturers: Cemex, Dixon-Marquette, Holcim, and LaFarge (in no particular order),
2. ¾ " limestone course aggregate from South Central Wisconsin (Yahara Materials, Madison),
3. ¾" Igneous river stone coarse aggregate from Northwestern Wisconsin (Croell Redi-Mix, LaCrosse),
4. Natural river sand from South Central Wisconsin (Wingra Corp, Madison),
5. Grade 100 Ground Granulated Blast-Furnace Slag (Holcim Inc.),
6. Vinsol resin air-entraining agent (SikaLatex),
7. Water-base, resin-base curing compound (W.R. Meadows 1200 white series),
8. Water-base, wax-base curing compound (W.R. Meadows 1600 white series).

Primary data collected included: chemical compositions and fineness of GGBFS and cements, activation indices for cement and slag-cement combinations, compressive strength at 3, 7, 14, 28, 56 and 365 days, deicer scaling wash-off to 100 freeze-thaw cycles. The strength testing considered 1 brand of Grade 100 GGBFS, 4 brands of Type I portland cement and 2 types of coarse aggregate. The deicer testing considered 1 brand of Grade 100 GGBFS, 2 brands of Type I portland cement, 2 types of coarse aggregate, and 4 different curing regimes. The four curing regimes were wet curing for 14 days followed by 14 days in laboratory dry air (approx. 70°F and 30% to 50% RH), 28 days of laboratory dry air exposure, application of a resin-based curing compound followed by 28 days of laboratory dry air exposure and application of a wax-based curing compound followed by 28 days of laboratory dry air exposure. Strength tests were based on five test replicates and deicer scaling tests were based on three test replicates.

Findings and Conclusions

The results showed significant differences in performance of GGBFS concrete depending on the combination of GGBFS, cement, aggregate and curing regime. While GGBFS can be used successfully to produce strong and durable concrete, the outcome depends on a combination of chemical and production factors whose differences may not be readily apparent.

The delayed strength development associated with GGBFS was also exhibited in the results of this research. The time required to achieve a traffic-opening compressive strength of 3000 psi at least doubles when 30% or more GGBFS replaces portland cement. The results can be summarized as follows with regards to the time required to achieve the 3000 psi strength:

- 0% GGBFS: range 3 to 5 days, average 4 days with limestone coarse aggregates, average 3 days with igneous coarse aggregates

- 30% GGBFS: range 4 to 13 days, average 7 days with limestone coarse aggregates, average 8 days with igneous coarse aggregates
- 50% GGBFS: range 6 to 16 days, average 10 days with limestone coarse aggregates, average 11 days with igneous coarse aggregates
- At 40°F, 0% GGBFS range: 11 to 18 days, 30% GGBFS range: 20 to 40 days, and 50% GGBFS range: 31 to 49 days.

The WisDOT required curing period for GGBFS concrete prior to opening to traffic when cylinder tests are not available, were not conservative in all the test cases for this data based on w/cm =0.45.

Deicer scaling results depended primarily on the level of GGBFS usage, the curing regime and the cement brand. These results were quantified based on scaling wash off in grams per square meter of exposed surface. The literature suggests that after 56 cycles, wash-off quantities of 50 g/m^2 or less would be considered very good, 500 g/m^2 or less good and 1000 g/m^2 or more would be considered unacceptable.   The results can be summarized as follows for wash-off levels we recorded at 50 cycles:

- 0% GGBFS: range 14 g/m^2 to 1101 g/m^2, moist curing best
- 30% GGBFS: range 156 g/m^2 to 1298 g/m^2, best curing depends on cement brand
- 50% GGBFS: range 355 g/m^2 to 2022 g/m^2, air dry curing best

The deicer scaling wash-off results suggested that 50% GGBFS usage would result in unacceptable scaling in most circumstances. While 30% GGBFS usage resulted in greater scaling than 0% GGBFS, certain combinations of cement brand and curing provided good performance. Of most surprise, the curing compounds provided little benefit and the wax-based curing compound treatment was associated with some of the highest scaling results. Identifying the successful combinations of cement and curing treatment may not be easily predicted without qualification testing.

The scaling performance of GGBFS concretes appears to be related to carbonation of the surface layer prompted by the chemistry of the hydration process with exposure to $C0_2$. Significant improvements in scaling performance will only be achieved when this chemistry can be understood and controlled.

In summary, under certain conditions Grade 100 GGBFS can be used successfully in Wisconsin pavements. The seemingly minor variations that result from different mix constituents in OPC appear to be accentuated in GGBFS concrete. A 50% GGBFS cement replacement level usually results in unsatisfactory performance from primarily a scaling perspective.   A 30% GGBFS cement replacement level will often be acceptable but the outcome depends on constituents and curing methods used.

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1. Problem Statement

Federal transportation directives encourage the use of by-product materials and mandate open and unrestricted competition for alternative cementitious materials. The Environmental Protection Agency listed ground granulated blast furnace slag (GGBFS) as a recyclable material in the Federal Register in 1995. Listing on the Register allows GGBFS to be used in meeting the minimum recyclable material content on many federally funded projects. The federal directive essentially mandates that the marketplace must be open for listed materials and limiting use of products such as GGBFS cannot be done without strong cause.

2. Objectives and Scope of Study

Ordinary portland cement (OPC) concrete is a robust material that can be produced with a wide variety of materials and conditions, and still produce an engineered material that meets specific requirements. While GGBFS is widely used as a portland cement partial replacement it is not clear if it can be used in the robust manner associated with OPC. The objective of this study was to establish GGBFS concrete mix performance over a range of materials common to Wisconsin concrete paving. A set of recommendations that identify the performance tradeoffs and limits were an anticipated outcome of the study. The research plan sought to quantify the effects on GGBFS concrete of Type I portland cement from four different manufacturers, three GGBFS replacement levels, and two types of aggregate.

The research plan consisted of four tasks:

- Task 1: documentation of Midwest GGBFS experiences and a literature survey update,
- Task 2: monitoring of the variability of GGBFS composition,
- Task 3: strength gain and air void development with 4 different brands of Type I portland cement at varying temperatures, and
- Task 4: deicer scaling tests with two different cements and with four different curing methods.

The strength gain and deicer scaling tests used GGBFS as a replacement of portland cement at 0%, 30%, and 50% levels. Thirty mixes were evaluated in Task 2 regarding strength gain, and thirteen mixes were evaluated in Task 3 regarding deicer scaling. Freeze-thaw testing for the deicer scaling tests followed a modified ASTM C672 procedure where testing was extended to 100 cycles.

3. Background

The most observable impacts of using GGBFS are reductions in early compressive strength and deicer scaling resistance, and these impacts have been demonstrated in earlier research. The rate of early strength gain is inversely proportional to the amount of GGBFS (ACI 233R 2000). Ultimate strength of GGBFS concrete can be higher than OPC because the hydration of GGBFS is prolonged. Depending on the fineness of the GGBFS, the strength of GGBFS concrete will usually exceed OPC after 7 to 21 days according to previous research. Opinions differ on the optimal replacement to obtain highest strength and not all studies agree that GGBFS will result in higher ultimate strength. GGBFS levels generally range from 25% to 50% replacement of portland cement (Lim & Wee 2000, ACI 233R 2000, Hooton 2000, Aldea et al. 2000). Lim and Wee found that GGBFS concrete does not have superior strength to OPC, regardless of GGBFS level or fineness (Lim & Wee 2000). The deicer scaling resistance decreases as GGBFS level increases (Soric-Corin & Aitcin 2002). The reduced scaling resistance is thought to occur because of an increase in a carbonation layer on the concrete surface caused by the GGBFS (Stark & Ludwig 1997). The poor deicer performance of GGBFS concretes prompted the Ontario Ministry of Transportation to limit GGBFS usage to 25% replacement of portland cement (Hooton 2000).

ACI 223R states that chemical composition of the GGBFS, alkali concentration of the system, glass content of the GGBFS, fineness of GGBFS and cement, and temperature during hydration affects the cementitious nature of the GGBFS (ACI 233R 2000). The chemical composition of the GGBFS is fixed by the steel making process, varies little and is not of major importance (Hooton 2000). Alkalis are needed to break down the glassy structure of the GGBFS allowing the hydration to begin (ACI 233R 2000). Although the GGBFS needs to be rapidly quenched to achieve a glassy structure, the particles do not need to be 100% glass for the system to be reactive. Fineness of the GGBFS is a major determinant in compressive strength of concrete. The compressive strength increases as the fineness is increased (Miura & Iwaki 2000) and is greatly affected by temperature during the early stages of hydration. Low curing temperatures significantly reduce compressive strength, and high temperatures accelerate strength gain (ACI 233R 2000, Hooton 2000, Miura & Iwaki 2000, Escalante-Garcia & Sharp 2001).

A literature review of pertinent research examined articles published from 1997 to 2004 and the main points have been highlighted in the discussion above. A bibliography of these articles can be found in Appendix I and synthesis of the literature is presented in Appendix II.

4. Methodology and Testing Regime

4.1 General

The methodologies of the four tasks of the research followed accepted procedures with some modifications to enhance the level of meaningful data. **Task 1** established the recent experiences with GGBFS in paving concrete to supplement previously published research. A survey of neighboring Midwestern state DOT's was conducted to identify potential commonality of field and performance problems. **Task 2** was targeted at assessing the variability of GGBFS from one producer as delivered to a local ready-mix plant. The tests used to quantify variability were chemical composition of the portland cement and GGBFS (ASTM C114), slag-activity tests (ASTM C989), and particle size distribution. **Task 3** was directed at measurement of air dry shrinkage, hardened air void, and compressive strength test results. The tests conducted are summarized in Table 1. Thirty mixes were completed in Task 3. The mix variables were cement brand, curing temperature, GGBFS replacement level, and aggregate type. The purpose of **Task 4** was to quantify the deicer scaling resistance of the GGBFS concrete utilizing the tests listed in Table 2. Task 4 had similar mix variables to Task 3, except that only two cement brands were used in the test matrix. The mix proportions for both Tasks 3 and 4 were based on WisDOT Grade A and Grade A-S designs. A water-cementitious material ratio (w/cm) of .45 was used with a target air content of 6% ± ½ %. Task 4 tests included slump, plastic air content, unit weight, and deicer scaling resistance according to ASTM C672. Table 3 shows the test matrix for the deicer scaling blocks.

Table 1. Summary of Tests Conducted in Task 3

Test	**Frequency**	**Applicable ASTM Standard**	**Curing Conditions**	**Age of Concrete at Tests (days)**
Slump	1 per batch	C143	None	0, fresh
Plastic Air Content	1per batch	C231	None	0, fresh
Unit Weight	1 per batch	C138	None	0, fresh
Air Void Analysis	1 per batch	C457	14 day wet	NA
Air Dry Shrinkage	3 per mix	C490, C157 modified	14 day wet	up to 120 days
Compressive Strength	2 per batch and 4 per mix	C39	Wet cured until tested	3, 7, 14, 28, 56, 365

Table 2. Summary of Tests Conducted in Task 4

Test	**Frequency**	**Applicable ASTM Standard**	**Curing Conditions**	**Age of Concrete at Tests (days)**
Slump	1 per batch	C143	N/A	0, fresh
Plastic Air Content	1per batch	C231	N/A	0, fresh
Unit Weight	1 per batch	C138	N/A	0, fresh
Deicer Scaling Resistance	3 per batch	C672	Varies	28

Table 3. Test Matrix for Deicer Scaling Tests in Task 4 (specimens per cement type per aggregate type)

Curing Method	GGBFS Replacement Level 0%	30%	50%
Air dry curing	3 specimens	3 specimens	3 specimens
Standard wet curing (14 days)	3 specimens	3 specimens	3 specimens
Wax based curing compound	3 specimens	3 specimens	3 specimens
Resin based curing compound	3 specimens	3 specimens	3 specimens

4.2 Materials

Materials were selected based on their pertinence to Wisconsin concrete paving operations. The following materials were used:

1. Type I cement from four manufacturers: Cemex, Dixon-Marquette, Holcim, and LaFarge (in no particular order)
2. ¾ in. limestone course aggregate from South Central Wisconsin (Yahara Materials, Madison)
3. ¾ in. igneous river gravel coarse aggregate from Northwestern Wisconsin (Croell Redi-Mix, LaCrosse)
4. Natural river sand from South Central Wisconsin (Wingra Corp, Madison)
5. Grade 100 Ground Granulated Blast-Furnace Slag (Holcim Inc.)
6. Vinsol resin air-entraining agent (SikaLatex)
7. Water-base, resin-base curing compound (W.R. Meadows 1200 white series)
8. Water-base, wax-base curing compound (W.R. Meadows 1600 white series)

The fine and coarse aggregate gradations are shown in Appendix III. Chemical compositions of the portland cement and GGBFS can be found in Appendix IV. The portland cements were randomly assigned a letter from A to D so that the results were anonymous.

All materials were used as provided by the manufacturer except for the aggregates. Aggregates were oven-dried for a minimum of 24 hours and allowed to cool to ambient temperature before use. This additional step was taken to have maximum control over the aggregate water content. During mix design the amount of water needed to achieve a w/cm (water to cementitious material) ratio of 0.45 was adjusted by the amount of water absorbed by the aggregates.

Limestone and igneous aggregates were used in this study and both meet Wisconsin Department of Transportation requirements for No. 1 stone (WisDOT 1996). The two types of aggregates were representative of the igneous river stones that predominant in concrete construction in the northern portion and the limestone aggregate most common in the southern portion of the state.

Aggregate absorptions were measured as part of the mix design effort of the research in general compliance with ASTM C127 and C128. The limestone coarse aggregate had an absorption value of 2.86% and the igneous aggregate had a value of 1.35%. The absorption test was also performed on the fine aggregate and produced a result of 0.85%.

4.3 Mix Design and Specimen Preparation

The concrete mixing was conducted at ambient lab conditions in two phases. Phase one and phase two were completed by different researcher staff but with overlap where the staff worked together briefly. All mixes were based on WisDOT Grade A and Grade A-S mix designs and the proportions for the mixes are listed in Table 4. Phase one involved preparing and testing the ambient lab temperature and cold temperature compressive strength cylinders, shrinkage specimens, and hardened air void specimens all using cement A. Also included were the scaling block mixes for cement A. Phase two covered ambient lab temperature mixes for cements B, C, and D. A set of scaling blocks was completed using cement C. The air void prisms and the shrinkage prisms were placed under wet burlap and covered with plastic for twenty-four hours before being demolded and placed in a wet room.

To quantify the scaling resistance of GGBFS, the deicer scaling blocks were cast according to ASTM C672 for cements A and C at three replacement levels and with two aggregate sources. The scaling blocks were then subjected to four distinct surface curing regimens. An additional batch for cement C was provided that utilized limestone aggregate and 30% replacement to explore a variation in curing time. A total of thirteen mixes were completed.

Four curing regimes were researched for each mix as follows:

1. air dry curing,
2. 14 day wet curing, 14 days ambient air dry curing,
3. wax based curing compound applied 45 minutes after finishing followed by ambient indoor conditions for 28 days,
4. resin based curing compound applied 45 minutes after finishing followed, by ambient indoor conditions for 28 days, and

Two additional curing regimes were used as follows:

5. A single mix with 30% GGBFS and limestone aggregate was subject to 14-days wet curing as before but with an extended period of indoor curing prior to be subject to deicer freeze/thaw tests. The additional curing involved places the blocks at ambient lab conditions for an additional 28 days for a total of 56 days of curing. This was done to observe the effect of increased hydration on the deicer scaling resistance.
6. The influence of 40°F curing conditions was pursued for strength and shrinkage testing specimens only. Specimens were cast at 40°F and maintained at 40°F for up to 56 days or until testing was complete. Strength cylinders were left in their molds with a plastic cap and maintained at 40°F until their day of testing.

The batches in both phases were prepared at a w/cm ratio of 0.45 and an air content of 6% ± ½ %. A vinsol resin air-entraining agent from one manufacturer and one shipment was used for all mixing. Fresh air content was measured according to ASTM C231. The aggregate correction factor for both the limestone and the igneous aggregates was 0.5%. When air content was not achieved the mix was dispatched, and the mix was redone.

Table 4. Mix Proportions by Aggregate Type (lbs per cubic yard).

	Limestone Mix			Igneous Mix		
	GGBFS Replacement			GGBFS Replacement		
Material (lb/yd^3)	0%	30%	50%	0%	30%	50%
Cement	564	397	286	564	397	286
GGBFS	0.0	167	286	0.0	167	286
Fine Agg.	1250	1242	1237	1250	1242	1237
Course Agg.	1874	1863	1858	1874	1863	1858
Water	254	254	257	254	254	257
Water Adjusted	319	319	321	289	289	292

4.4 Test Methods

Tests of the hardened concrete were conducted according to the applicable ASTM standards listed in Table 1 and Table 2. Compression tests of 4 in. diameter by 8 in. long concrete cylinders were completed according to ASTM C39. The cylinders were wet cured until testing. Tests of five specimens from each mix were done at 3, 7, 14, 28, 56, and 365 days. Prior to testing the specimens were sulfur capped in accordance with ASTM C617.

The compressive strengths were adjusted for the relatively small differences in air content using the equation introduced by Popovics (1998) and shown in Eqn 1. This formula is based on the assumption that for every 1% increase in air content, the compressive strength is reduced by 5%.

Corrected Strength = Measured strength x (1 - .05 x 6%)/(1 - .05 x Air Content %)
(Eqn. 1)

Determination of length change due to drying shrinkage followed ASTM C490. Three specimens per mix were made using a 10 in. long mold with a 4 in. x 4 in. cross section. Each mold was filled and then vibrated until the determination was made that the majority of entrapped air was released, which usually resulted in a vibration time of ten to fifteen seconds. After the concrete was vibrated, the surface of the specimens was troweled smooth. Specimens were wet cured for a period of fourteen days. After wet curing, the specimens were moved to a climate controlled conditions with a temperature of 73.5 °F and a relative humidity of 50%.

Measurements were taken immediately after demolding, every day at an age of 14 to 21 days, every 3 days from 21 to 57 days, and every 7 days from 57 days to 120 days. Specimens that did not show a general cessation of length change at 120 days were measured every 7 days until that determination could be made. Length change was measured using a vertical length comparator. To find the length change at a given age the average percentage length change for the three specimens was found. Equation 2 from ASTM C157 was used to find the percentage length change for each prism.

ΔL_x = (CRD – initial CRD)/G *100 (Eqn. 2)

ΔL_x = length change of specimen at any age, %

CRD = difference between the comparator reading of the specimen and the reference bar at any age

G = gage length (10 in.)

Two length comparators were used in measuring the specimens. A second comparator became necessary because some of the specimens were stored offcampus at the WisDOT central office laboratories. Instead of transporting the dial comparator to the secondary location, a digital comparator was purchased to measure the specimens located at the secondary facility. A discrepancy was found to exist between the standard bar of the dial comparator and the standard bar of the digital comparator. To reconcile this difference, Eqn 2 was adjusted. The value of .0101 was added to CRD when using the new digital comparator. This value was used because the reference bar supplied with the digital comparator was .0101 longer than the reference bar of the older comparator.

Scaling blocks were prepared in accordance with ASTM C672. Approximately seven days before initiating the brine pond testing, foam dikes were applied to the specimens to retain the deicer solution. Compressed air was used to clear the surface of the blocks of any loose debris. Next, two one-quarter inch passes of a foam insulation material were applied to the specimen surface.[1] The foam insulation expanded after application so the width of the foam was approximately one inch wide around the perimeter of the block surface and three-quarter inches high. After the insulation set, eight minutes according to the manufacturer, a quarter inch bead of caulk was applied to the inner and outer surfaces of the dike where the insulation met the concrete.[2] The caulk was applied to provide an extra barrier to water leakage.

Scaling tests were performed according to ASTM C672 with two modifications. First, C672 specifies that the specimens be subject to a temperature of -18°C for 16 to 18 hours and 23°C for 6 to 8 hours. This was modified to -20°C for 20 hours and 30°C for 4 hours. The room in which the specimens were placed contained 120 blocks and was only capable of achieving temperatures of -20°C, so the modification was made to ensure that the blocks were completely frozen. A thermocouple was placed inside one block, and the observed values confirmed that the blocks were frozen below 0°C consistently. The higher temperature of 30°C was specified so that the blocks would adequately thaw in the four hour time period. All specimens were subjected to 100 freeze/thaw cycles. The brine pond was a 4% solution of sodium chloride.

[1] Great Stuff window and door insulating foam sealant, manufactured by Dow Chemical.

[2] DAP Alex Plus Acrylic Latex Caulk plus Silicone, a window and door indoor/outdoor caulk.

The second alteration to ASTM C672 involved the evaluation of the concrete surface. In addition to the numerical ratings required by C672, the block surface was flushed with water and the scaled off material was collected in a No. 200 sieve. The material was then oven-dried and weighed. Also, the area of each block was measured so that the data could be normalized by area.

Hardened air void analysis was completed by American Petrographic Services according to ASTM C457, Procedure A. The researcher provided one-inch wide samples cut from the middle of each concrete prism. One sample from each batch was sent for testing. Two additional samples were sent from a 50% GGBFS limestone aggregate mix and a 50% GGBFS igneous aggregate mix to observe the agreement between multiple samples of the same mix.

5. Experience Survey and Test Results

5.1 Experience Survey

A survey of neighboring state departments of transportation was conducted to determine their experiences with GGBFS. Table 5 displays the grades and amounts of GGBFS commonly used in the region; as well as the restrictions that are placed on the use of GGBFS in concrete pavements. Along with this information, contacts were asked what assumptions and problems were common when using GGBFS. The most common assumptions were the well known perceptions of GGBFS usage that concrete will have lower early strength and lower scaling resistance. Knowing that these problems were possible, steps were taken to prevent them. While no contacts reported actual problems with scaling and cracking, most did increase the mix water content to properly coat the GGBFS particles and obtain comparable workability to OPC concrete. In addition to a higher water demand, many also experienced longer set times when GGBFS was used. Detailed comments on assumptions, usage, problems, and solutions are located in Appendix V. In reviewing the survey comments, it was determined that this study was looking at appropriate variables when using GGBFS as a partial replacement for portland cement; therefore, mix designs and test procedures remained as originally proposed.

Table 5. Experience Survey Summary

State	MN	IA	IL	IN
Common GGBFS Grade Used	100 & 120	100 & 120	100 & 120	100 & 120
Common GGBFS Replacement Level	35%	35%	25%	30%
Use Restriction (Temperature)	Up to contractor (< 90^{o}F)	None	> 40^{o}F	None
Use Restriction (Seasonal)	Up to contractor	None	None	Apr. 1 – Oct. 15

5.2 Variability of Grade 100 GGBFS in the Madison Area

As with cement, the fineness of GGBFS is a key factor in its hydraulic reactivity. The strength development of concrete increases with an increase of GGBFS fineness (Miura and Iwaki, 2000). There are three grades of GGBFS; 80, 100, 120, which are said to relate primarily to Blaine fineness values of approximately 400, 600, and 800 m^2/kg. It was concluded from the regional survey that Grade 100 and 120 are both commonly used, and Grade 80 generally is not permitted in highway paving projects. In addition to use in highway paving, Grade 100 GGBFS was and continues to be used by ready-mix operations in the Madison area. We obtained permission to gather monthly samples of the GGBFS supplied to the ready-mix operation. A monthly timeframe provided a reasonable sampling of different shipments of GGBFS arriving at the plant. The grade 100 GGBFS was the same brand used in this research. To distinguish the ready-mix samples from the single shipment of GGBFS used in the laboratory research described herein, we refer to the laboratory shipment of GGBFS as GGBFS-rs. The "rs" is short for research slag.

Five monthly samples along with a sample of the GGBFS-rs were sent to a commercial laboratory to analyze their particle characteristics, which are summarized in Table 6. The first three samples were sent for particle and chemical analyses. Since these showed little variation, testing was temporarily discontinued.. We then resumed further sampling approximately 6 months later. To stay within budget we terminated particle analysis and conducted Blaine fineness testing in-house. These results are also shown in Table 6. The last rows of Table 6 show the standard deviation and the standard deviation divided by the mean. These statistics indicate that the physical properties of the GGBFS shipped to the Madison area vary little over time.

Table 6. Particle Analysis for Monthly GGBFS Samples

Monthly Sample	Size at 50% Passing (µm)	Mean Diameter (µm)	Specific Surface Area (m^2/kg)
GGBFS-RS	9.71	13.32	557
08/2002	9.24	12.75	528
09/2002	9.76	13.82	560
10/2002	9.39	12.65	558
03/2003	8.20	11.16	599
07/2003	8.60	11.88	570
01/2004	NA	NA	563
02/2004	NA	NA	579
03/2004	NA	NA	562
04/2004	NA	NA	558
05/2004	NA	NA	569
Standard Deviation	0.63	0.96	17.2
(Std Dev.)/Mean	7%	8%	3%

In addition to fineness, the chemical composition of GGBFS contributes to its effectiveness as a cementitious material. Chemical analyses were conducted on the same samples sent to a commercial laboratory for particle analysis. Each chemical constituent was recorded as a percent by weight (Table 7). As with the GGBFS fineness, there were no outstanding chemical variations for the monthly samples. The range for any given compound never exceeded 1.2%. When comparing the monthly samples to GGBFS-rs it was observed that GGBFS-rs had the least lime (CaO), a major component in strength development. Given the small variation in chemical composition observed here, no additional chemical tests were undertaken in the study.

Table 7. Chemical Analysis for Monthly Samples

	Weight (%)					
Chemical Constituent	**Aug-02**	**Sep-02**	**Oct-02**	**Mar-03**	**Jul-03**	**GGBFS-RS**
SiO_2	37.60	37.73	37.40	37.48	37.50	37.22
Al_2O_3	8.21	7.73	7.71	7.15	7.48	7.78
Fe_2O_3	0.69	1.01	0.95	0.82	0.77	1.01
CaO	38.89	38.02	38.37	39.22	38.55	37.62
MgO	11.30	10.95	10.70	10.19	10.46	10.98
SO_3	2.59	2.23	2.51	2.58	2.42	2.53
Na_2O	0.28	0.27	0.26	0.38	0.32	0.30
K_2O	0.40	0.37	0.35	0.28	0.34	0.33
TiO_2	0.45	0.46	0.48	0.74	0.80	0.43
P_2O_5	<.01	<.01	<.01	<0.01	<0.01	<0.01
Mn_2O_3	0.48	0.55	0.54	0.51	0.53	0.56
SrO	0.04	0.04	0.04	0.04	0.04	0.04
Cr_2O_3	0.01	0.01	0.01	0.01	0.01	0.01
ZnO	<.01	<.01	<.01	<0.01	<0.01	<0.01
L.O.I. (950^{o}C)	-0.08	0.33	0.31	0.06	0.38	0.56
Total	100.11	99.70	99.63	99.46	99.59	99.39
Alkalis as Na_2O	0.55	0.52	0.50	0.56	0.55	0.52

Regardless of GGBFS particle and chemical properties, ultimately the interaction of GGBFS with portland cement hydration chemistry will determine the acceptability of the GGBFS. ASTM C989 grades GGBFS according to its hydraulic activity. To determine hydraulic activity the strength of mortar containing GGBFS was compared to the strength of mortar without GGBFS at 7 and 28 days. For each monthly sample from August 2002 to July 2003, 3 sets of 6 mortar cubes were cast along with a set of 6 reference mortar cubes using cement A. The monthly samples from January 2004 to May 2004 used cement B, cement C, and cement D. One set of 6 cubes were cast for each month and cement. Three cubes from each set were tested at 7 days and the other three were tested at 28 days. A set of cubes was compared to the reference set made on the same day to calculate the slag activity index (SAI).

Results of the SAI tests indicate low variability (Figures 1 to 4). Each monthly sample deviates little from the 7-day and the 28-day averages for a given cement. GGBFS from this particular ready-mix company appears to be quite stable when examining the results from particle distribution, specific surface, chemical composition, and activity tests.

Figure 5 shows the differences in SAI among the different cements at 7 and 28 days for the research GGBFS. Most specimens passed the minimum average SAI set forth by ASTM C989 for 7 and 28 days. Cement A did not meet the minimum 28-day SAI of 95. It was near the minimum with a SAI of 92. The 7-day averages were close to each other for all the cements. Separation by compressive strength of the cements occurred at 28 days. Cement C was clearly the strongest followed by B, D, and A. The reference cements met the strength requirements of ASTM C989 as shown in Table 8 but only Cement C fell in the required alkali range.

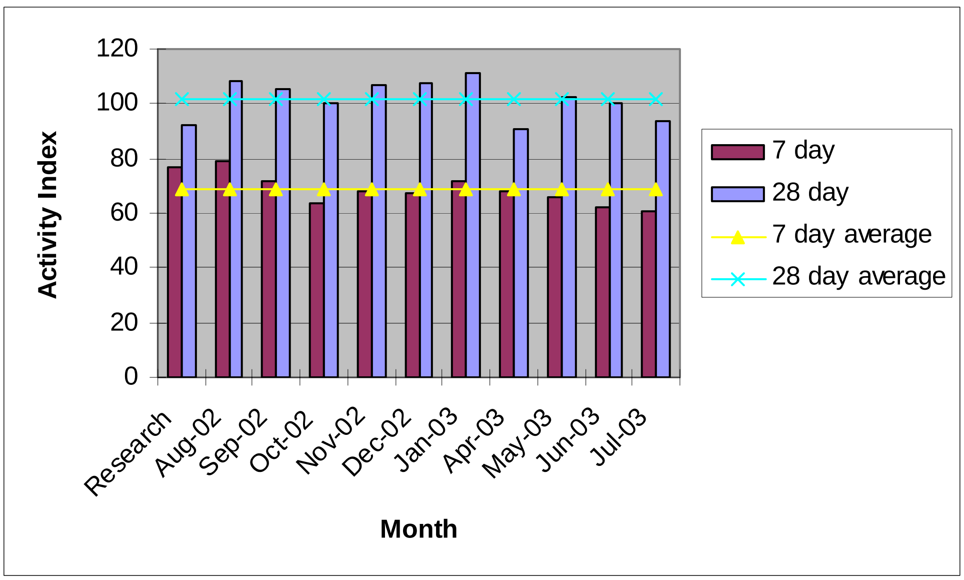

Figure 1. Monthly Sample Slag Activity Index for Cement A

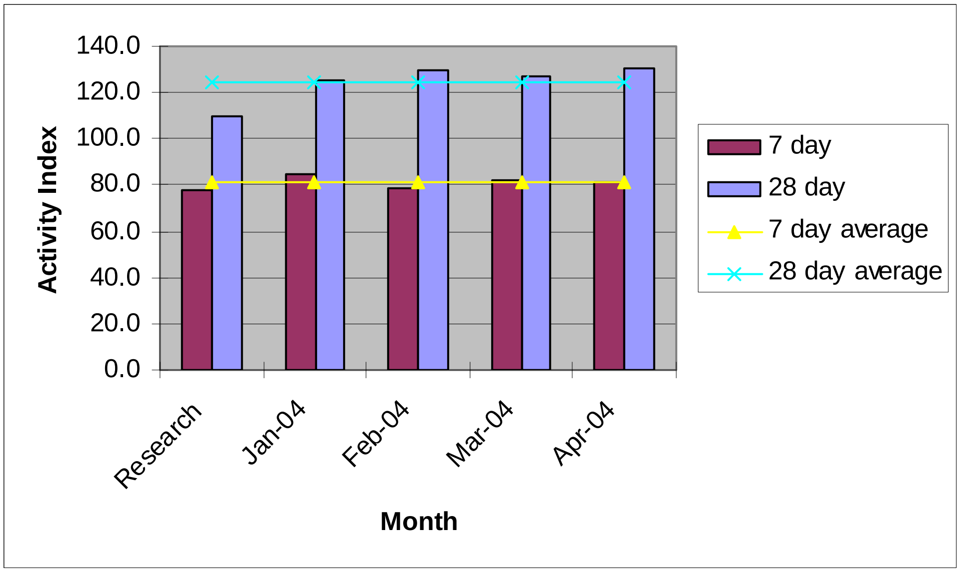

Figure 2. Monthly Sample Slag Activity Index for Cement B

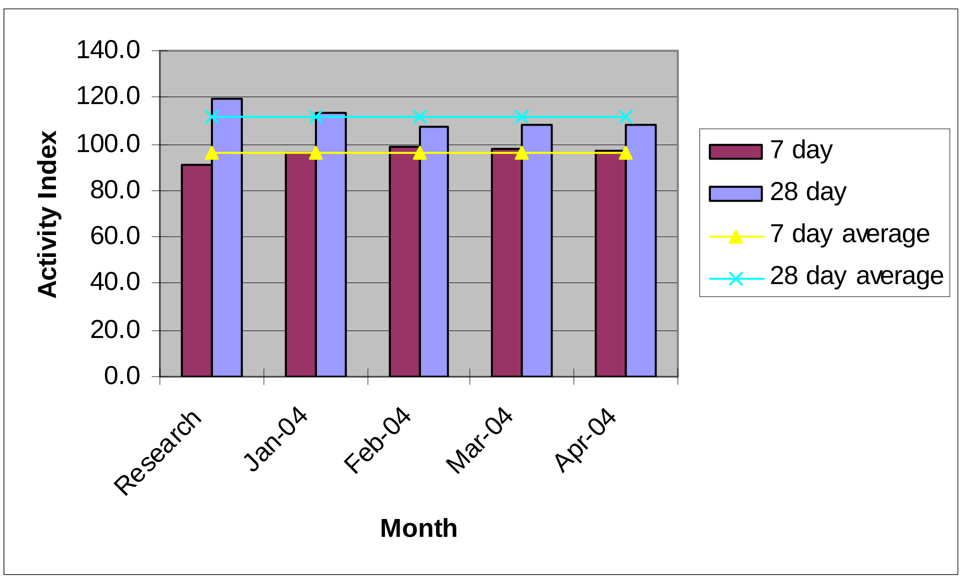

Figure 3. Monthly Sample Slag Activity Index for Cement C

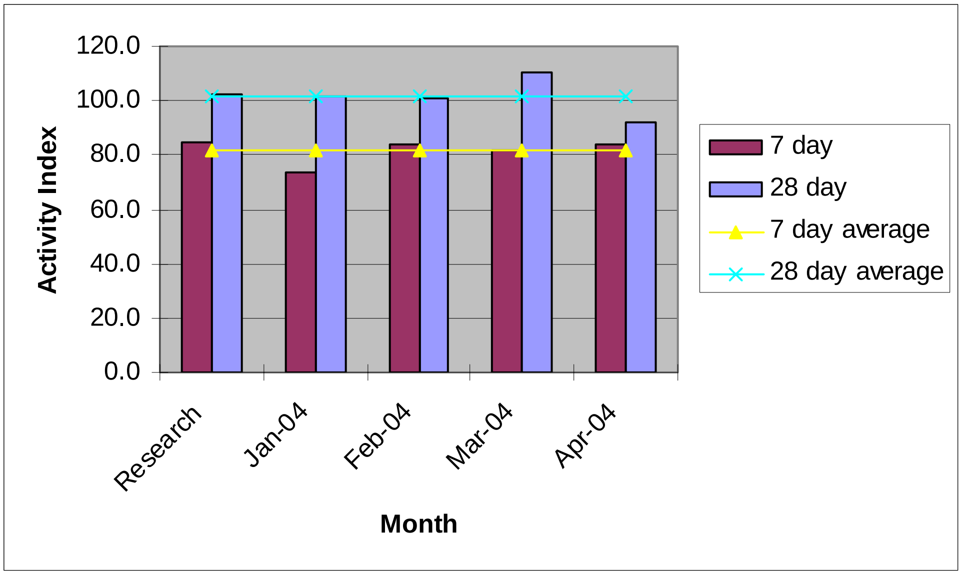

Figure 4. Monthly Sample Slag Activity Index for Cement D

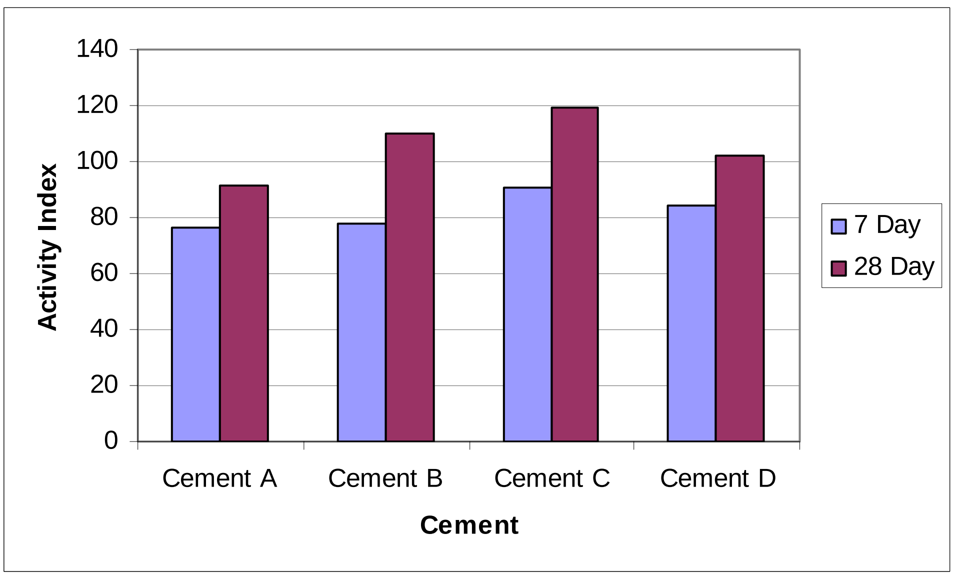

Figure 5. Research GGBFS Slag Activity Index for Each Cement

Linear correlations were developed between 7-day cylinder compressive strength and 7-day SAI and 28-day SAI, and 28-day cylinder compressive strength with 7-day SAI and 28-day SAI. The correlations were established for each individual cement and then the results were averaged. These values are shown in Table 9.

Table 8. ASTM C989 Reference Cement Requirements Compared

Cement	Total Alkalies (Na_2O+0.658K_2O)	28 day Compressive Strength, psi
Cement A	0.42	5475
Cement B	1.02	5080
Cement C	0.83	5177
Cement D	0.54	6520
ASTM C989 Reqmnt	0.60 to 0.90	5000 (min)

Table 9. Average Correlations Between Activity Index and Compressive Strength

7d Activity w/ 7d Strength	0.89
7d Activity w/ 28d Strength	0.86
28d Activity w/ 7d Strength	0.84
28d Activity w/ 28d Strength	0.82

5.3 Plastic Concrete Results

Table 10 and Table 11 show the composition of the compression mixes and the deicer scaling mixes, respectively. Mixes 8 through 13 were produced at a temperature of 40°F. All other mixes were performed at ambient laboratory conditions. The results of the slump, unit weight, and plastic air content tests are given in Table 10. The hardened air void results are shown for comparison purposes.

The fresh concrete properties had little variability according to Table 10. All plastic air contents were within the 6% ± ½ % range. Slumps were 2.25 in ± 1.25 in and unit weights were 141.5 lb ± 4 lb. Fresh air content and hardened air content did not agree for most of the batches. Concrete tends to lose air entrainment as it is allowed to sit so this could explain the disagreement when the hardened air content is lower than the fresh air content. Higher hardened air contents may be due to entrapped air that resulted from inadequate vibration of the test prisms.

5.4 Compression Test Results

The compressive results suggest that different brands of cement perform differently with different aggregate types. Figure 6 shows the compressive strength regression line over a period of one year for the four different brands of cement, no GGBFS and limestone aggregates. Figure 7 presents similar data with igneous aggregates. With the limestone aggregates (Fig. 6) the different cements respond considerably different at early age but converge to a similar compressive strength at one year. In contrast, with igneous aggregates the cements tend to track parallel over time and the cement that was weakest at early age with limestone aggregates proved to be the strongest with igneous aggregate. There were no obvious chemical or fineness attributes that distinguished the performance characteristics of the different cements.

Table 10. Composition of the Compression Mixes

Mix	% Slag	Cement	Aggregate	Slump in.	Weight lb/ft^3	Air Content %	Hardened Air Content %
2	0	A	Limestone	3.00	140.4	6.1	8.3
3	30	A	Limestone	3.00	138.4	6.3	8.2
4	50	A	Limestone	2.75	137.6	6.4	9.4
5	0	A	Igneous	1.50	145.4	6.1	4.5
6	30	A	Igneous	2.25	144.8	5.4	7.9, 7.1
7	50	A	Igneous	2.00	142.4	6.0	5.1
8-1	0	A	Limestone	2.50	140.8	6.5	9.0
8-2	0	A	Limestone	2.50	139.6	6.5	-
9-1	30	A	Limestone	1.25	142.8	5.5	-
9-2	30	A	Limestone	2.25	141.7	6.1	-
9-3	30	A	Limestone	2.50	141.4	5.7	5.9
10-1	50	A	Limestone	2.50	141.5	6.5	-
10-2	50	A	Limestone	2.75	142.4	6.0	-
10-3	50	A	Limestone	2.50	143.0	5.6	4.9
11-1	0	A	Igneous	2.75	143.5	5.5	6.2
11-2	0	A	Igneous	2.75	141.8	5.6	-
11-3	0	A	Igneous	2.50	143.8	5.5	-
12-1	30	A	Igneous	2.25	141.6	6.3	-
12-2	30	A	Igneous	2.75	144.8	6.5	7.8
13-1	50	A	Igneous	2.50	143.4	6.0	5.9
13-2	50	A	Igneous	2.75	143.7	5.7	-
13-3	50	A	Igneous	2.75	143.1	6.0	-
14	0	B	Limestone	2.50	143.2	6.0	5.5
15	30	B	Limestone	3.50	144.0	6.0	5.8
16	50	B	Limestone	3.25	142.0	5.9	5.7
17	0	B	Igneous	2.75	144.8	6.1	4.4
18	30	B	Igneous	2.75	143.2	6.0	6.2
19	50	B	Igneous	2.75	142.4	6.3	5.6
20	0	C	Limestone	2.25	143.2	6.1	4.7
21	30	C	Limestone	2.75	142.4	5.7	5.2
22	50	C	Limestone	2.75	141.6	5.9	5.6
23	0	C	Igneous	2.25	145.2	6.0	6
24	30	C	Igneous	2.63	144.4	5.7	4.9
25	50	C	Igneous	1.50	144.8	5.6	5.6
26	0	D	Limestone	3.25	143.2	6.0	6.1
27	30	D	Limestone	2.50	141.6	5.6	6.4
28	50	D	Limestone	2.75	140.4	6.1	5.8
29	0	D	Igneous	2.50	144.4	5.7	4.8
30	30	D	Igneous	2.00	144.4	6.0	5.6
31	50	D	Igneous	2.00	142.4	5.8	4.6

Table 11. Composition of the Deicer Scaling Mixes

Mix Number	% Slag	Cement	Aggregate
2A	0	Cement A	Limestone
3A	30	Cement A	Limestone
4A	50	Cement A	Limestone
5A	0	Cement A	Igneous
6A	30	Cement A	Igneous
7A	50	Cement A	Igneous
0C	0	Cement C	Limestone
1C	30	Cement C	Limestone
2C	50	Cement C	Limestone
3C	0	Cement C	Limestone
4C	30	Cement C	Limestone
5C	50	Cement C	Limestone
6C	30	Cement C	Limestone

Figures 8 through 11 show the strength trends on a cement brand basis for specimens where GGBFS was introduced. It is evident from examination of these figures that there are not clear rules or trends that apply to universally to the strength results from all four cements. The trends are dependent on cement type and vary amongst cement types. Three day strengths of OPC were 1-1/2 to over 2 times those containing 50% GGBFS with the 30% GGBFS specimens' strengths failing in between. GGBFS concretes will often meet and exceed the OPC strengths but this occurs anywhere from about 14 days to over 56 days. In some cases, even after 1 year, the GGBFS concrete has not achieved the strength of the OPC. Generally at early ages, the concrete containing the igneous aggregate achieved higher strength and at later ages the concrete with the limestone aggregate was often stronger.

The lower early strength of GGBFS concrete compared to OPC is well known, but the rate of the strength gain is not well defined. Hogan and Meusel concluded that overall strength is lower through three days, but higher after that as compared to OPC (Hogan & Meusel 1981). Average compressive strength data from this study shown in Table 12 refutes this finding. In no case did the GGBFS concretes' strength gain come close to this. ACI Committee 233R states that Grade 100 GGBFS should have greater strength than regular concrete after 21 days (ACI 233R 2000). The data supports this in about half of the cases.

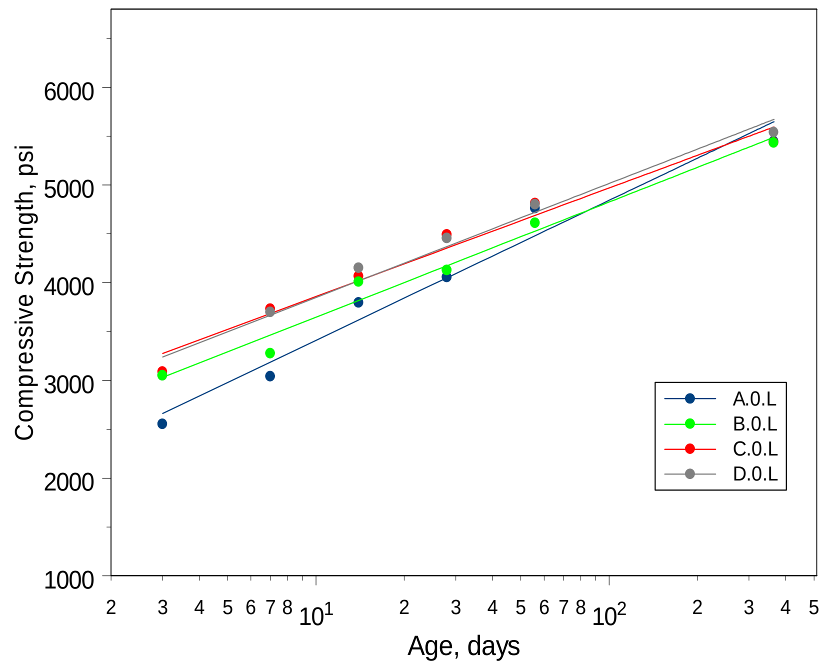

Figure 6. Compressive strength trends of OPC for four different cements (A, B, C, D) and limestone aggregates

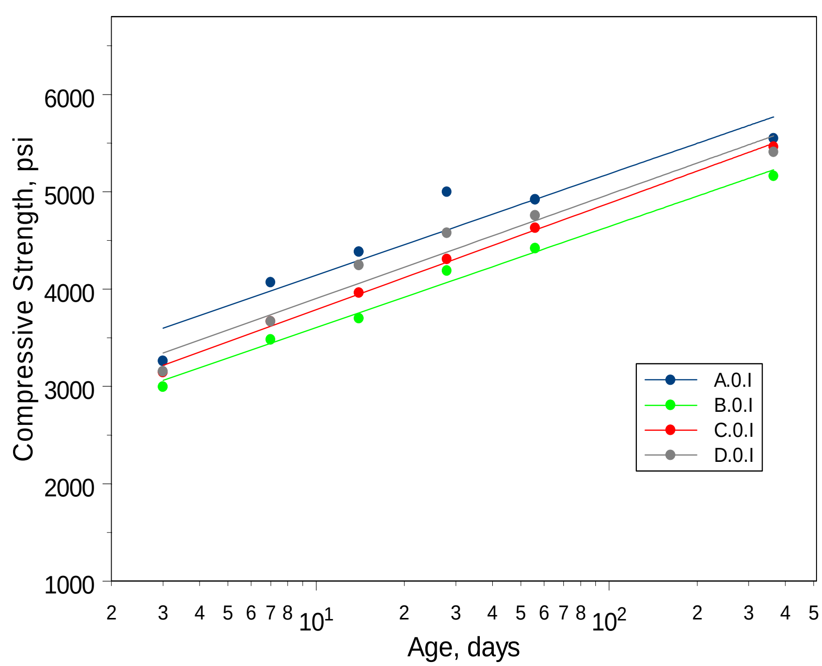

Figure 7. Compressive strength trends of OPC for four different cements (A, B, C, D) and igneous aggregates

CEMENTA

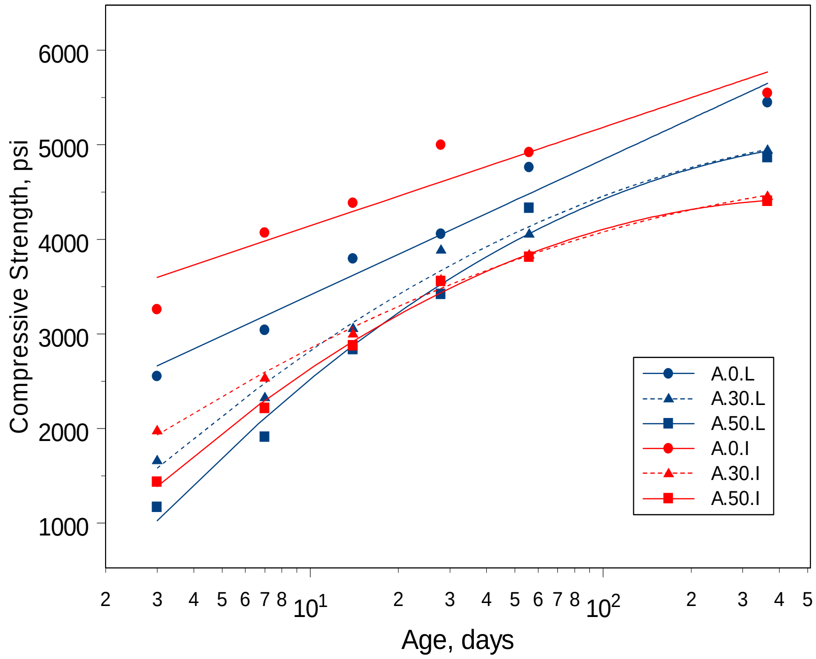

Figure 8. Compressive strength trend line specimens using cement A (Key: cement type, GGBFS %, aggregate type (L= limestone, I = igneous)

CEMENTB

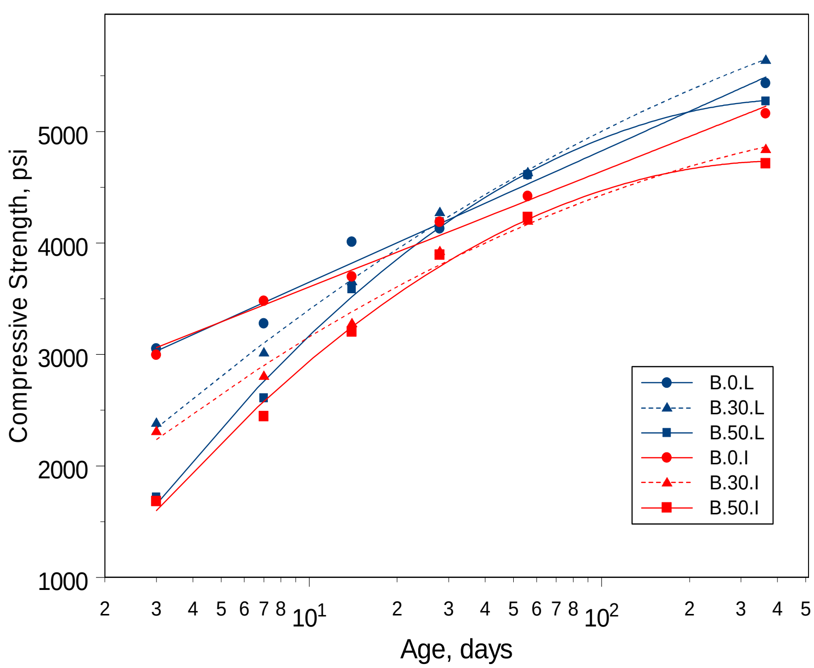

Figure 9. Compressive strength trend line for specimens using cement B (Key: cement type, GGBFS %, aggregate type (L= limestone, I = igneous)

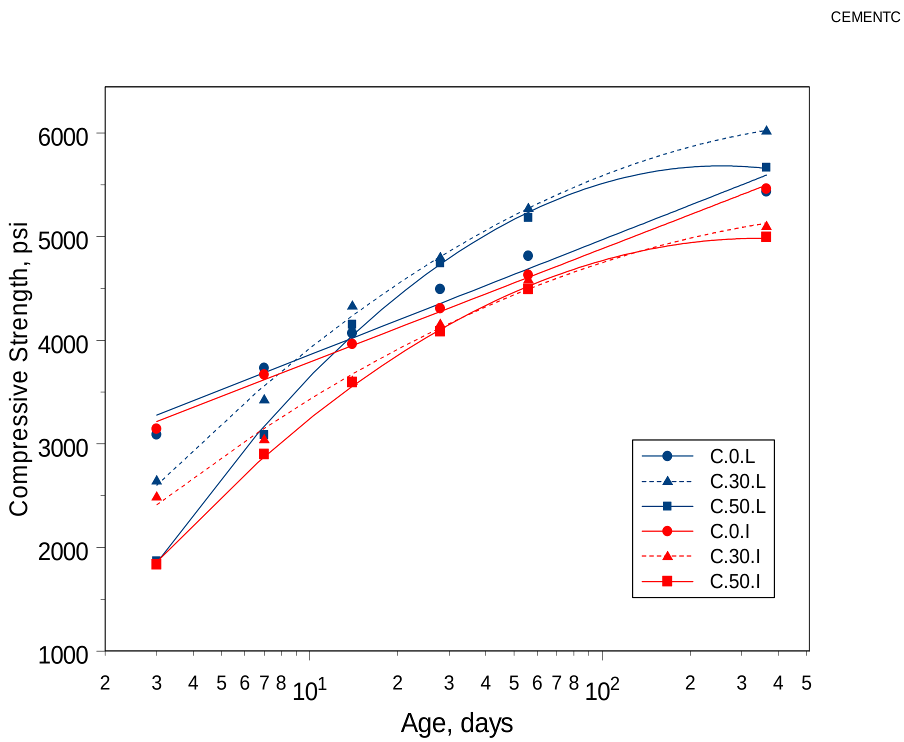

Figure 10. Compressive strength trend line for specimens using cement C (Key: cement type, GGBFS %, aggregate type (L= limestone, I = igneous)

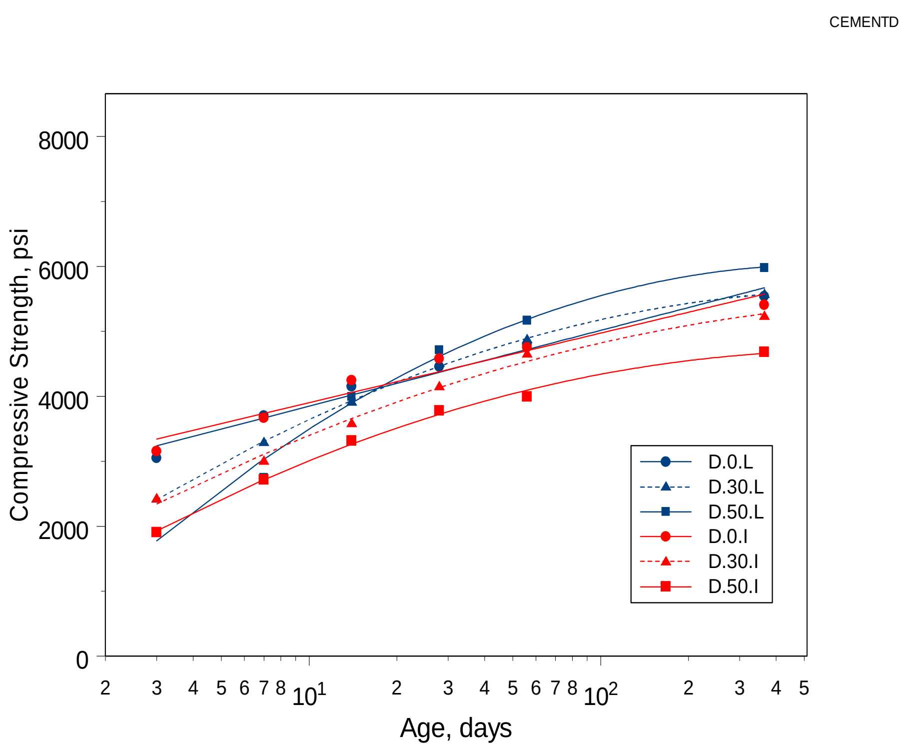

Figure 11. Compressive strength trend line for specimens using cement D (Key: cement type, GGBFS %, aggregate type (L= limestone, I = igneous)

Table 12. Average compressive strength for all cement and aggregate types

	Compressive Strength		
Test Day	0% GGBFS	30% GGBFS	50% GGBFS
3	3036	2290	1690
7	3579	2931	2579
14	4038	3552	3445
28	4400	4159	4047
56	4713	4514	4477
365	5428	5226	5072

The rate of strength gain is important because it strongly influences the delay between completion of paving and road opening. WisDOT requires a compressive strength of 3000 psi to open to traffic (DOT 2004). Table 13 shows by age when the mixes would reach the standard of 3000 psi assuming laboratory curing conditions. In approximately 3 days, OPC will reach 3000 psi providing confirmation of provision 415.3.17.1 of the WisDOT Construction Specification. For A-S mix designs, WisDOT requires 7 days before opening to traffic. Table 13 reveals an average of 7-8 days to reach 3000 psi for 30% GGBFS levels but with certain cement brands the strength development may be slower. GGBFS usage at 50% generally requires 10 to 11 days on average to reach 3000 psi but again with certain cements the time required exceeded 2 weeks. The current 7-day requirement in WisDOT Construction Specification 415.3.17.1 is insufficient for 50% levels of GGBFS but it is important to note all of these results are based on w/cm = 0.45.

WisDOT Construction Specification provision 415.3.17.1 uses a 0.6 factor for curing days at 40°F. This would translate to 5 days of required curing for OPC. Table 12 suggests that 11 to 18 days are required for strength to reach 3000 psi. For Grade A-S mixes the 40 degree curing temperature would dictate 12 days of curing but Table 13 suggests that 20 to 49 days is required to reach 3000 psi depending on GGBFS level and aggregate type.

Table 14 confirms the late strength gain characteristics of GGBFS. After 3 days all the GGBFS concretes had less than 95% of the ordinary concrete strength. However, only three cases of less than 95% strength existed after 56 days. The averaged compressive data for each mix can be found in Appendix VI.

5.5 Deicer Scaling Resistance Test Results

ASTM C672 stipulates that a visual rating of the scaling blocks be made every 5 cycles to determine the effect of deicing agents. The visual rating is a subjective measure with a scale of 0 to 5. This prompted development of a procedure that weighed the scaled material washed from the block surface to give an objective measure of the deicing chemical effects. Recall that 2 different cements were used with 3 different cement replacement levels with 2 different aggregates and 4 different curing regimes. Tables 15 and 16 list the actual levels of wash-off scaling material after 50 and 100 cycles of freeze-thaw exposure respectively. As

Table 13. Time Needed to Reach Traffic Opening Strength of 3000 psi

Cement Manufacturer	Aggregate	Replacement Level	Time to Reach 3000 psi (days)
A	Limestone	0%	5
		30%	12
		50%	16
	Igneous	0%	3
		30%	13
		50%	16
B	Limestone	0%	3
		30%	6
		50%	9
	Igneous	0%	3
		30%	8
		50%	11
C	Limestone	0%	3
		30%	4
		50%	6
	Igneous	0%	3
		30%	6
		50%	8
D	Limestone	0%	3
		30%	5
		50%	7
	Igneous	0%	3
		30%	6
		50%	10
Average	**Limestone**	**0%**	**4**
		30%	**7**
		50%	**10**
	Igneous	**0%**	**3**
		30%	**8**
		50%	**11**
A (40°F)	Limestone	0%	11
		30%	20
		50%	31
	Igneous	0%	18
		30%	40
		50%	49

Table 14. Compressive Strength as a Percentage of the 0% Replacement

Cement	Aggregate	Replacement	3 Days	7 Days	14 Days	28 Days	56 Days	365 Days
A	Limestone	0%	100%	100%	100%	100%	100%	100%
		30%	64%	75%	78%	88%	83%	91%
		50%	50%	68%	81%	86%	98%	89%
A	Igneous	0%	100%	100%	100%	100%	100%	100%
		30%	61%	62%	68%	72%	78%	80%
		50%	44%	54%	66%	71%	78%	79%
A 40°F curing	Limestone	0%	100%	100%	100%	100%	100%	NA
		30%	57%	69%	76%	86%	94%	NA
		50%	47%	55%	52%	66%	89%	NA
A 40°F curing	Igneous	0%	100%	100%	100%	100%	100%	NA
		30%	58%	64%	76%	69%	75%	NA
		50%	62%	51%	67%	66%	79%	NA
B	Limestone	0%	100%	100%	100%	100%	100%	100%
		30%	78%	92%	91%	104%	100%	104%
		50%	56%	80%	89%	101%	100%	97%
B	Igneous	0%	100%	100%	100%	100%	100%	100%
		30%	77%	81%	89%	94%	95%	94%
		50%	56%	70%	87%	93%	96%	91%
C	Limestone	0%	100%	100%	100%	100%	100%	100%
		30%	86%	92%	106%	107%	110%	111%
		50%	61%	83%	102%	106%	108%	104%
C	Igneous	0%	100%	100%	100%	100%	100%	100%
		30%	79%	83%	91%	96%	99%	93%
		50%	58%	79%	91%	95%	97%	91%
D	Limestone	0%	100%	100%	100%	100%	100%	100%
		30%	79%	89%	94%	101%	102%	101%
		50%	62%	74%	96%	106%	108%	108%
D	Igneous	0%	100%	100%	100%	100%	100%	100%
		30%	77%	82%	84%	91%	98%	97%
		50%	60%	74%	78%	83%	84%	87%

a point of reference, at 56 cycles, wash-off quantities of 50 g/m^2 or less would be considered very good, 500 g/m^2 or less good and 1000 g/m^2 or more would be considered unacceptable (Johnston 1994). Visual ratings and the raw data can be found in the Appendix VII. These data allow several important comparisons including; the influence of limestone coarse aggregate versus igneous river gravel, the influence of different curing membranes and regimes, the influence of percentage of GGBFS, and the influence of cement brand.

Table 15. Cumulative Wash Off (g/m^2) at 50 Cycles

		Limestone		Igneous			Limestone
Curing Method	**0%**	**30%**	**50%**	**0%**	**30%**	**50%**	**30%- 56 day cure**
Cement A							
Wax	274	802	2022	1101	1298	1060	-
Resin	181	552	1905	480	1379	2015	-
Dry	203	614	1577	169	336	566	-
Moist	14	1638	2604	61	578	1444	-
Cement C							
Wax	116	639	819	142	292	919	344
Resin	45	266	789	134	235	481	259
Dry	18	222	355	105	168	373	206
Moist	28	156	735	136	288	689	92

Table 16. Cumulative Wash Off (g/m^2) at 100 Cycles

		Limestone		Igneous			Limestone
Curing Method	**0%**	**30%**	**50%**	**0%**	**30%**	**50%**	**30% - 56 day cure**
Cement A							
Wax	383	945	2096	1225	1439	1240	-
Resin	303	724	2039	558	1549	2428	-
Dry	290	809	1746	197	401	753	-
Moist	29	1904	2731	76	640	1512	-
Cement C							
Wax	134	705	887	155	377	1028	390
Resin	60	319	934	165	389	567	306
Dry	38	344	561	120	234	518	235
Moist	32	193	882	158	397	740	213

Influence of Aggregate Type on Deicer Scaling Resistance

There was not a consistent pattern between aggregate type and scaling across both cement brands and all curing regimes. Limestone aggregate was associated with the least scaling in exactly 50% of the situations with cement A. When considering cement C, limestone was associated with 70% of the situations with least scaling. The magnitudes of the differences between washoff associated with different curing regimes were more pronounced with cement A than with cement C. Generally, limestone aggregate GGBFS mixes experienced significantly more scaling relative to their OPC (0% GGBFS) control mix as GGBFS quantity increased compared to the same with igneous aggregate mixes. This may be partly because OPC mixes containing igneous aggregate showed higher levels of scaling. The interactions between scaling and aggregate type appear to be complex and further investigation would be needed to determine the precise causal mechanisms.

Influence of Curing Method on Deicer Scaling Resistance

Figure 12 shows washoff quantities for OPC. As expected, moist curing generally provided a surface with the minimum or near-minimum scaling. Surprisingly, open air curing provided a surface nearly as durable as wet curing. The curing compounds provided no observable benefit and appeared to be detrimental compared to providing no surface conditioning (air dry). Wax curing appeared to perform the worst and sometimes significantly worse than the other curing regimes. In any event, scaling was minor for the 0% GGBFS specimens as indicated by the previous points of reference.

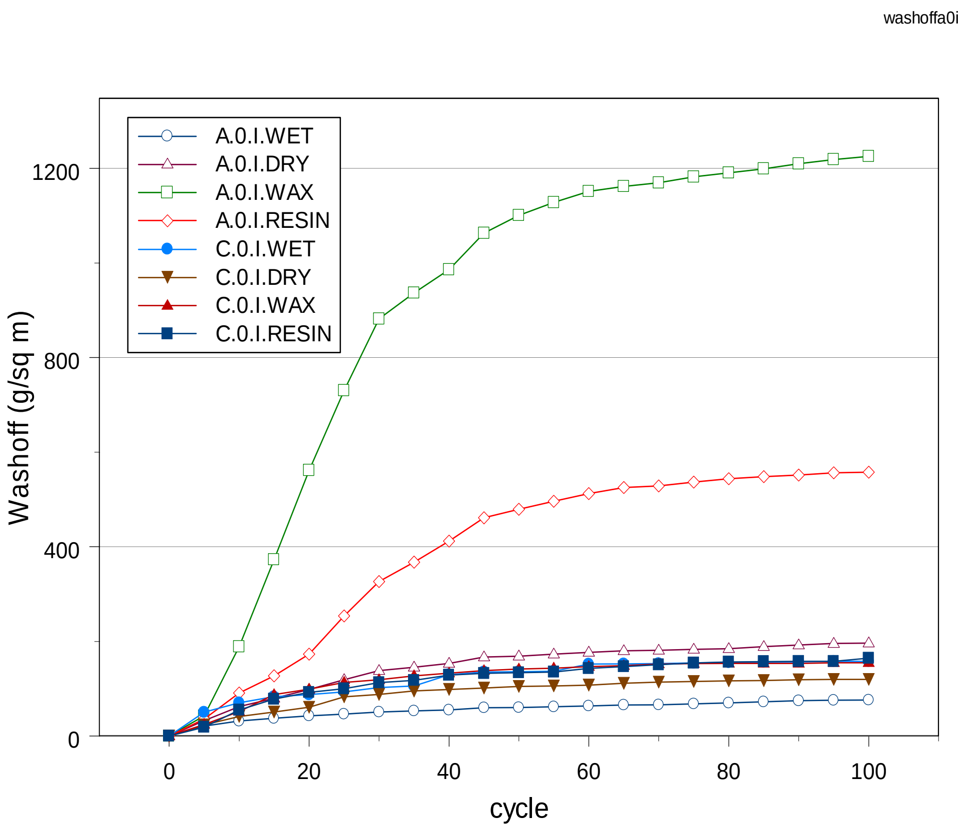

Figure 12. Washoff quantities for OPC (igneous coarse aggregate) subject to different curing regimes.

Curing method at the 30% level did not have a pronounced effect on cement C, but cement A responded differently (Figure 13 and Table 15). Wax curing and resin curing performed poorly with the igneous aggregate, and moist curing did not provide protection when limestone was used. In general air dry curing resulted in the least scaling (Fig. 13). Scaling in the 500 to 1000 g/m^2 range indicates unsatisfactory surface deterioration.

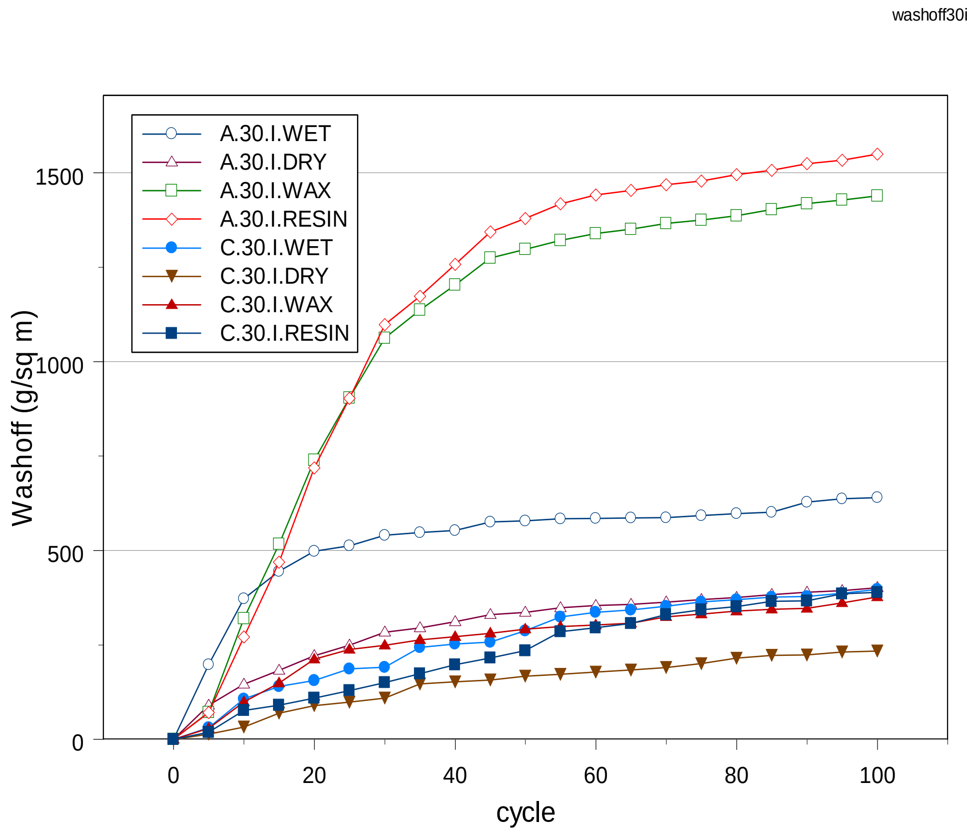

Figure 13. Washoff quantities with 30% GGBFS and igneous coarse aggregate subject to different curing regimes.

Curing methods had a pronounced effect with the 50% GGBFS concretes (Fig 14 and Table 15). Air dry curing was the most effective method to minimize scaling and surprisingly wet curing provided the least protection. As Fig. 14 illustrates, the curing method became more important as replacement level increased, which was also found by Afrani and Rogers (Afrani & Rogers 1994). The deicer scaling performance of GGBFS concretes depended significantly on the combination of the replacement level, aggregate type, cement type and the type of curing. By most standards, GGBFS concrete scaling became unacceptable regardless of conditions at the 50% replacement level. It is apparent that the mechanism that provides a surface resistant to scaling is fundamentally different for GGBFS concrete.

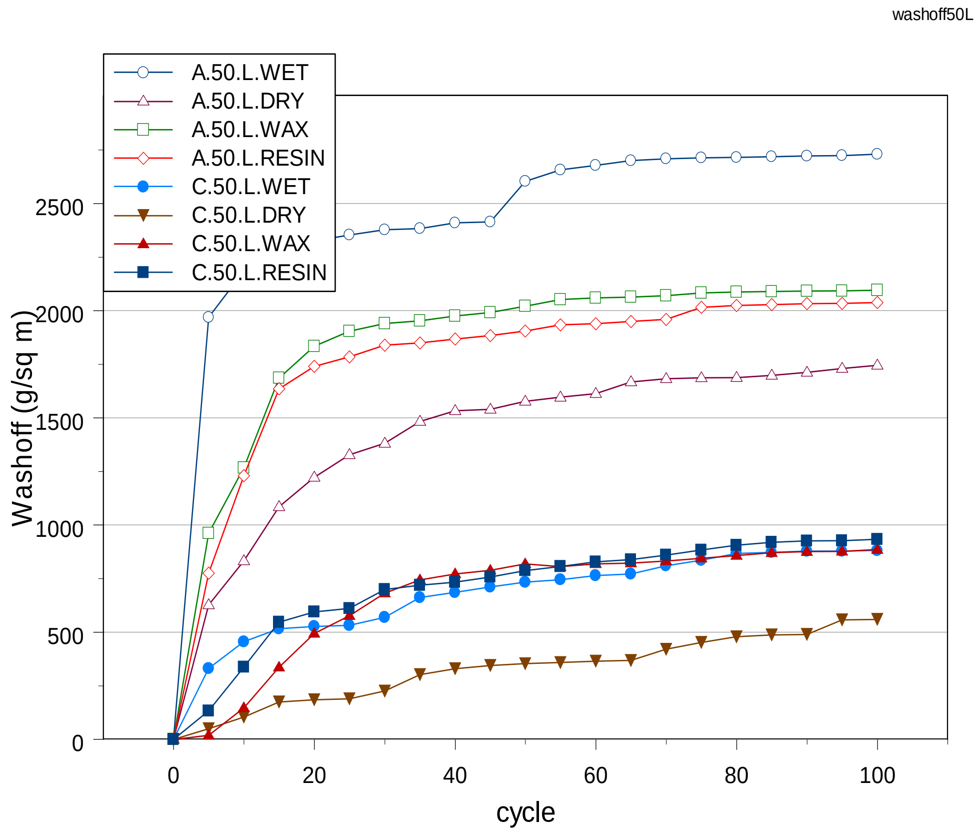

Figure 14. Washoff quantities with 50% GGBFS and igneous coarse aggregate subject to different curing regimes.

Influence of Replacement Level on Deicer Scaling Resistance

As GGBFS replacement rises so does the severity of scaling (Fig. 15, Tables 15 and 16). This held true across different aggregates, curing methods, and cements. At 30% GGBFS usage, scaling was 3 to 6 times greater than OPC scaling when limestone aggregates were used and approximately 2 times greater than OPC scaling when igneous aggregates were used. At 50% replacement, scaling was 7 to 20 times that of OPC concrete with limestone aggregates and 3 to 6 times that of OPC with igneous aggregates.

Many curves of the GGBFS concretes showed an initial period (0 to 10 cycles) of little scaling followed by a high rate of scaling and then the curves settled into a rate that resembled the ordinary portland concretes. This trend is most visible at the 50% replacement level and seems to depend upon use of GGBFS because it did not occur with most of the 0% mixes. The scaling phenomenon of the GGBFS mixes has been addressed by Stark and Ludwig. They also showed that GGBFS concrete had a high rate of scaling followed by a break point in which the scaling became similar to that of ordinary concrete. They hypothesized that the heavy initial scaling was caused by carbonation of the concrete surface during initial curing. The calcium carbonate in the outer surface was dissolved by the action of the frost and chloride of the deicer salt which causes the scaling. After the carbonated layer is dissolved, the high rate of scaling subsides. SEM tests support this hypothesis (Stark

& Ludwig 1997). Also, the depth of carbonation increases as GGBFS content increases (Sakai et al 1992). This further supports the hypothesis because the rate of initial scaling increases as GGBFS level rises.

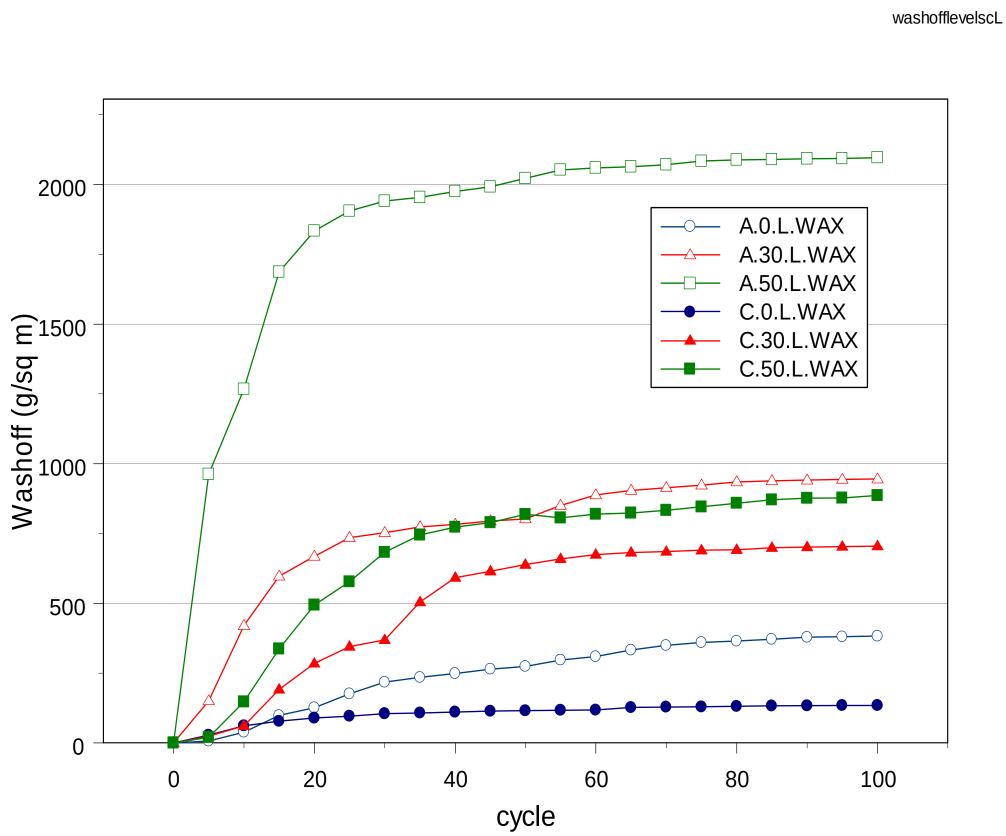

Figure 15. Washoff quantities for different levels of GGBFS Usage with limestone coarse aggregate and wax-based curing compound.

A hypothesis was made that the percentage of carbonate would increase as the level of deicer scaling increased for a given curing regimen. Therefore, it was expected that the percentage of carbonate ($CaCO_3$) would be highest in the wax cured washoff followed by decreasing concentrations in wet cured, resin cured, and air cured washoff. Thermogravimetric analysis was employed to determine the level of carbonate present in these different samples. The results of the analysis showed the percentage of calcium carbonate in the resin cured washoff was 29.4% followed by 29.2% in the wax cured washoff, 28.9% in the air cured washoff, and 26.0% in the wet cured washoff. These differences did not support the hypothesis offered.

Several explanations are possible. First, the differences in the amount of washoff may not have been statistically significant. However, an ANOVA of test reveals that the differences among the curing regimens are significant at the 0.05 level. Second, the washoff for each curing method was taken from a jar that contained material from all 100 cycles. Washoff samples used in the analysis likely consisted of washoff at different points in the 100 cycle test program. Washoff taken earlier in the test program may have had higher carbonate content than those taken later. The most probable explanation is that a carbonate content of approximately 25% to 30% corresponds to the highest amount of carbonate that can be present due to the limiting reactant calcium hydroxide ($Ca(OH)_2$). A new question should then be asked. If not the concentration of carbonates, then does the depth of carbonation affect the severity of scaling? Stark and Ludwig answered this question in the affirmative (Stark & Ludwig 1997). Unfortunately, the quantification of carbonation depth was not included as part of this study, nor could it be included after the conclusion of the deicer scaling block tests. Further studies should investigate the carbonation depth in an attempt to confirm Stark and Ludwig's findings and to expand upon them by including examination of novel curing regimens.

Influence of Portland Cement Brand

Cement A scaled more than cement C in almost every case (Figures 12 to 15, Table 15). When GGBFS was used, cement C performed better than cement A for all combinations tested. The differences observed are difficult to explain as no literature could be located where the cement manufacturer was a variable in a study. One must conclude that the hydration of the GGBFS is strongly influenced by the cement chemistry associated with subtle differences introduced by cement brand, combined with possible chemical interactions with aggregate coatings.

Influence of Curing Time on Deicer Scaling Resistance

One mix of 30% GGBFS with limestone aggregate was cast and cured for 56 days instead of 28 days to gauge the response of scaling resistance to curing time. From Fig. 16 and Tables 15 and 16, it appears that the advantage of extended curing time was minimal with one exception. Wax-based curing compound resulted in significantly less scaling with the longer cure time. While the increased curing time allowed for additional hydration to occur, carbonation at the surface also occurred during this time. This combination likely caused the amount of scaling to be similar to the 28-day cured blocks.

Degree of hydration which is characterized by strength is often given as a broad indication of carbonation. However, this does not give an adequate representation of the outer surface of concrete. Curing method gives a better indication of carbonation (Neville 1996). Extended wet curing of 27 days has been shown by others to be superior to 14 days of wet curing which is given by the standard (Saric-Coric & Aitcin 2002).

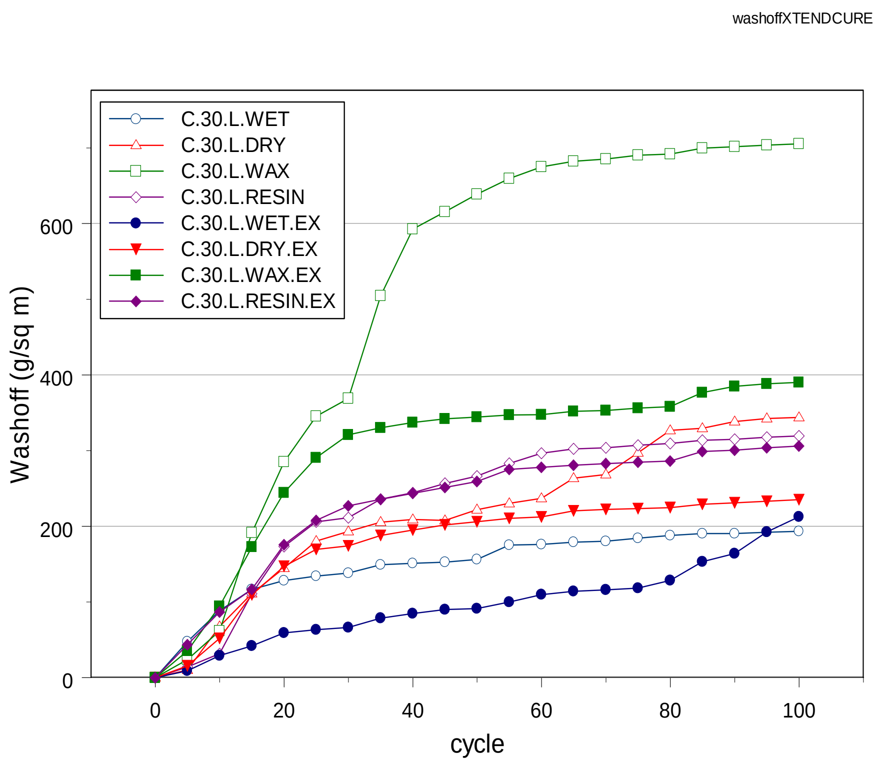

Figure 16. Washoff contrasting 28 days versus 56 days of curing with 30% GGBFS and limestone aggregate.

5.6 Air Dry Shrinkage

The 30% GGBFS mixes at ambient temperatures tended to have largest air dry shrinkages as shown in Tables 17 and 18. Shrinkages from the 0% and 50% GGBFS mixes showed mixed results.

The level of drying shrinkage with GGBFS concrete showed mixed results in the literature. ACI Committee 233 cites studies which show that GGBFS causes increased shrinkage and in another of its cited studies the shrinkage is found to be decreased or comparable to OPC (ACI 233 2000). Malhotra found that shrinkage was comparable for GGBFS and OPC concretes while Hogan and Meusel found increased shrinkage for GGBFS concretes (Malhotra 1989, Hogan & Meusel 1981). One study even found that shrinkage decreased as GGBFS increased (Sakai et al. 1992).

Tables 17 and 18 present the results from this study. With room temperature curing, shrinkage in GGBFS concrete was somewhat higher than OPC. The prisms stored at 40^{o}F were only measured for 56 days; therefore, the effects of GGBFS replacement level and drying temperature were compared based on 56-day measurements for cement A. The effect of GGBFS on drying shrinkage was dependant on the exposure temperature as seen in Table 17. From these results, there does not appear to be a clear benefit or harm in shrinkage related to the use of GGBFS.

Table 17. 56-Day Shrinkage for Cement A and Cold Mixes

Cement Manufacturer	Aggregate	GGBFS %	56-day Shrinkage %	Percentage of 0% mix
A	Limestone	0	-0.0343	100.0%
		30	-0.0427	124.3%
		50	-0.0473	137.9%
A	Igneous	0	-0.0393	100.0%
		30	-0.0673	171.2%
		50	-0.0400	101.7%
A - Cold mix	Limestone	0	-0.0207	100.0%
		30	-0.0180	87.1%
		50	-0.0233	112.9%
A - Cold mix	Igneous	0	-0.0267	100.0%
		30	-0.0163	61.2%
		50	-0.0293	110.0%

Table 18. 120-Day Shrinkage for Cement B, Cement C, and Cement D

Cement Manufacturer	Aggregate	GGBFS %	120-day Shrinkage %	Percentage of 0% mix
B	Limestone	0	0.0390	100.0%
		30	0.0490	125.6%
		50	0.0450	115.4%
B	Igneous	0	0.0423	100.0%
		30	0.0520	122.9%
		50	0.0400	94.6%
C	Limestone	0	0.0450	100.0%
		30	0.0530	117.8%
		50	0.0467	103.8%
C	Igneous	0	0.0497	100.0%
		30	0.0587	118.1%
		50	0.0460	92.6%
D	Limestone	0	0.0507	100.0%
		30	0.0750	147.9%
		50	0.0407	80.3%
D	Igneous	0	0.0583	100.0%
		30	0.0547	93.8%
		50	0.0387	66.4%

6. Guidelines on Use and Summary of Findings

The guidelines and predictive equations outlined below are based on the experiences of this study. Data from testing with grade 100 GGBFS and four brands of Type I cement with two types of aggregate was averaged and used to establish the equations. As a result there will be some variation from these equations particular conditions and materials, but the guidelines should be a representative guide of expected results.

6.1 Compressive Strength Guidelines

Mantel investigated the correlations between the chemical composition of cements and slags and their strength performance. After examining five GGBFS samples and eight cements he found that there was no clear correlation between chemical composition and performance (Mantel 1994). Mantel recommends that trial mixes be prepared to determine a portland cement's adequacy when used with GGBFS. To simplify this process the correlations of strength with the slag activity tests in section 5.2 were made.

In an attempt to establish general guidelines the results of all compressive tests at each replacement level were graphed. Figure 17 reveals that compressive strength for GGBFS concrete tend to be nonlinear as GGBFS level increases reflecting the slower strength development at early ages. At one year, the strengths of all concretes tend to converge but GGBFS levels tend to result in slightly lower strengths on average. The following equations represent the average strength as a function of age for w/cm =0.45:

$$CS_{0\%\,GGBFS} = -229 * (Log(Age))^2 + 1843 * Log(Age) + 2205 \qquad \text{(Eqn. 3)}$$

$$CS_{30\%\,GGBFS} = -418 * (Log(Age))^2 + 2713 * Log(Age) + 1031 \qquad \text{(Eqn. 4)}$$

$$CS_{50\%\,GGBFS} = -712 * (Log(Age))^2 + 3815 * Log(Age) - 22 \qquad \text{(Eqn. 5)}$$

Where CS = compressive strength in psi and Age = concrete age in days.

The following compressive strength guidelines can be formed from this study:

- Higher 7-day slag activity indices tend to indicate increased 7-day and 28-day strengths
- Limestone aggregate tended to produce slightly stronger concrete (10% or less).
- GGBFS should not be used at temperatures lower than 40°F as the time to achieve 3000 psi can be unreasonably long for w/cm = 0.45.
- 50% GGBFS concrete has comparable strength to 30% GGBFS concrete after 14 days with w/cm = 0.45.
- While there is delayed early strength development in GGBFS concrete, Grade 100 GGBFS concrete at cement replacement to 50% has comparable (but slightly less) strength to OPC after 56 days

strengthave

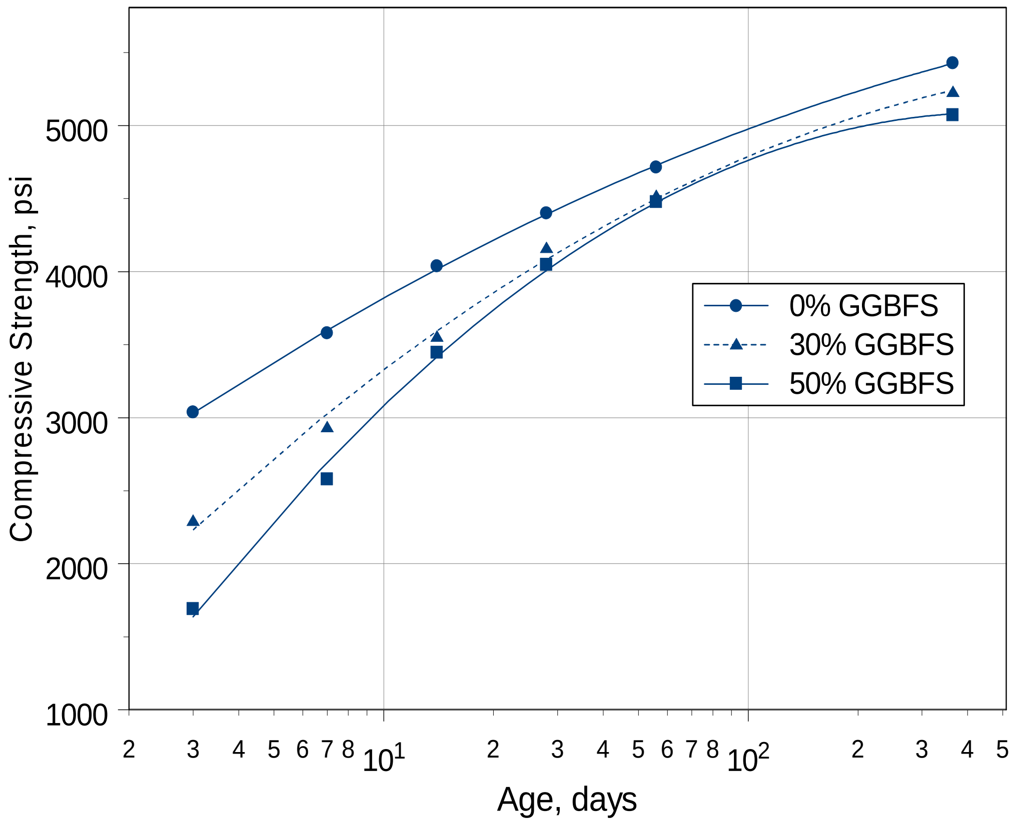

Figure 17. Average compressive strength for all specimens for given cement replacement levels.

6.2 Deicer Scaling Resistance Guidelines

Developing predictive methods for resistance to deicing chemicals encountered similar problems to those for compressive strength. Different brands of cement may react quite differently with GGBFS. As a result, the guidelines below are provided to define the performance of GGBFS concrete under ideal conditions.

- Wax curing compound performed poorly with GGBFS and should be avoided until the precise cause of the poor performance can be pinpointed or another study can show suitable performance.
- Resin-based curing compounds, Air Dry, and Moist curing can be used successfully with GGBFS, but the results depend on specific conditions.
- 30% GGBFS provided adequate resistance to deicing salts.
- 50% GGBFS should not be used where scaling can occur.
- Low GGBFS compressive strength may be an indication of scaling susceptibility.

- Doubling the curing time to 56 days provided only slight improvement in scaling resistance

6.3 Summary of Findings

The economic and environmental benefits of GGBFS have led to increasing use throughout the country as a road building material. Particularly relevant are the experiences of other Midwestern states, which use GGBFS as a 25% to 35% replacement for portland cement. This study tested Grade 100 GGBFS at the 0%, 30%, and 50% replacement levels and finds that the 30% replacement level effectively balances the properties of GGBFS and portland cement. The performance of GGBFS concrete will vary based on subtle differences in hydration chemistry as prompted by differing cement brands and possible reactions with aggregate coatings.

APPENDIX I – BIBLIOGRAPHY

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APPENDIX II – SYNTHESIS OF BIBLIOGRAPHY

Admixtures

Concept	References
In GGBFS concretes more air entrainer is needed to obtain same % air as PCC	Hooton, 2000 ACI 233R, 2000 Deja, 2003
Air entraining agent reduced shrinkage and improved workability with no negative effect on compressive strength in concretes with AAS as the only binder	Bakharev et al, 2000
Greater amounts of retarder will have a greater retarding effect as the proportion of GGBFS is increased	ACI 233R, 2000
Stabilizing very fine air bubbles in GGBFS concretes is difficult	Saric Coric & Aitcin, 2002

Chemistry & Hydration

Concept	References
Glass content does not have to be 100% for slag to be reactive	Hooton, 2000
GGBFS chemical composition is not of major importance in reactivity	Hooton, 2000
There is no correlation between chemical composition of slag and the hydraulic activity	Babu & Kumar, 2000
GGBFS lowers peak hydration temperature and extends the time needed to arrive at it	Alshamsi, 1997
GGBFS tends to increase propensity to bleeding, but particle size distribution of GGBFS has no relationship to bleeding characteristics	Olorunsogo, 1998
Initial hydration of GGBFS is slower than portland cement, but hydration increases as alkalis from the hydration of the cement break down the glassy GGBFS particles	ACI 233R, 2000
GGBFS concretes sets slower than portland cement concretes	Hogan, Meusel, & Spellman, 2001 Hooton, 2000

Curing and Curing Temperatures

Concept	References
GGBFS concrete is more susceeptible to poor curing conditions than ordinary concrete	ACI 233R, 2000
Steam curing and autoclaving decrease compressive strength compared to normal curing	Aldea et al, 2000
Degree of hydration for GGBFS increased as temperature increased from 10°C to 60°C	Escalante-Garcia & Sharp, 2001

GGBFS strength gain is retarded at low temperatures and accelerated at high temperatures	Hooton, 2000
Water, sealed, and air curings do not greatly affect 7 day compressive strength of GGBFS concrete, but the long-term strength of air cured GGBFS concrete is reduced at low temperatures	Miura & Iwaki, 2000

Alkali-Silica Reaction (ASR)

Concept	References
A higher level of alkalis can be tolerated as GGBFS content increases	Thomas & Innis, 1998
A minimum of 35% to 50% GGBFS replacement can control ASR	Hooton, 2000 Malvar et al, 2002 Duchesne & Berbe, 2001 ACI 233R, 2000
Alkali activated GGBFS concrete is more susceptible to alkali-aggregate reaction than ordinary portland concrete	Bakharev et al, 2001

Sulphate Resistance

Concept	References
High resistance to sulphate attack when GGBFS replacement exceeds 50%	ACI 233R, 2000
Beneficial effects of GGBFS have been demonstrated at 40% replacement	Hooton, 2000
GGBFS concrete provided good sulphate resistance when the alumina content of the GGBFS was less than 14%	Osborne, 1999
As Al_2O_3 content of the GGBFS rises so does the GGBFS level needed to increase sulphate resistance	Hooton, 2000

Chloride Penetration Resistance

Concept	References
GGBFS concrete has higher resistance to chloride-ion penetration than ordinary concrete	Hooton, 2000 ACI 233R, 2000 Osborne, 1999 Aldea et al, 2000
GGBFS replacement as low as 30% enhanced the resistance to chloride penetration as compared to ordinary concrete	Wee, Suryavanshi, & Tin, 2000

Shrinkage

Concept	References
50% GGBFS replacement has the largest autogenous shrinkage	Lim & Wee, 2000

Autogenous shrinkage increased as GGBFS fineness increased	Lim & Wee, 2000
Drying shrinkage of GGBFS concrete should be similar to ordinary concrete for a given aggregate volume and type	Hooton, 2000
Studies have shown mixed results with respect to drying shrinkage. Some studies reported increased shrinkage while others did not. The addition of gypsum will reduce the shrinkage of GGBFS concrete if that is a concern.	ACI 233R, 2000

Strength

Concept	References
For a given surface area, the more fine particles of GGBFS (< 3µm), the higher its early strength; the more 3 to 20µm particles, the higher the late strength	Wan et al, 2004
25% GGBFS replacement is optimal for strength under normal curing conditions	Aldea et al, 2000
Concrete with 50% GGBFS has similar strength to ordinary concrete	Aldea et al, 2000
Compressive strength decreases as GGBFS fineness decreases and as GGBFS replacement increases	Miura & Iwaki, 2000
Early strength (1-3 day) will be lower with GGBFS concrete under most conditions	Hooton, 2000
GGBFS concrete strength often exceeds normal concrete after 14 days except at high replacements	Hooton, 2000
High early strength can be obtained in GGBFS concretes with the addition of silica fume	Hooton, 2000
Greatest 28-day strength for GGBFS concretes occur with 40% to 50% replacement	ACI 233R, 2000
For early strength, the rate of strength gain is inversely proportional to the amount of GGBFS	ACI 233R, 2000
3-day strength of 50% GGBFS can be raised if total amount of binder is increased by 10%	Babu & Kumar, 2000
GGBFS concrete has early strength development problems at low temperatures (5°C) for surface area of 400 m^2 but not 800m^2	Miura & Iwaki, 2000
Grade 120 GGBFS give reduced strength at early ages (1-3 days) and higher strength after 7 days; grade 100 GGBFS gives increased strength after 21 days; grade 80 GGBFS gives reduced strength at all ages	ACI 233R, 2000
50% GGBFS replacement gives the highest compressive strength	Lim & Wee, 2000
Strength development of GGBFS was slower than ordinary concrete, regardless of GGBFS replacement levels and fineness	Lim & Wee, 2000

Deicer Scaling Resistance

Concept	**References**
Water curing GGBFS concrete for 27 days produced significantly better resistance than 14 days of water curing	Saric-Coric & Aitcin, 2002
As GGBFS content rises, the scaling resistance decreases	Saric-Coric & Aitcin, 2002
Polypropylene microfibres added to GGBFS concretes increased the scaling resistance at high replacement	Deja, 2002
Carbonation of the concrete surface causes lower scaling resistance in GGBFS concretes	Stark & Ludwig, 1997
Due to poor performance, the Ontario Ministry of Transporation limits GGBFS replacement to 25%	Hooton, 2000

APPENDIX III – AGGREGATE GRADATIONS

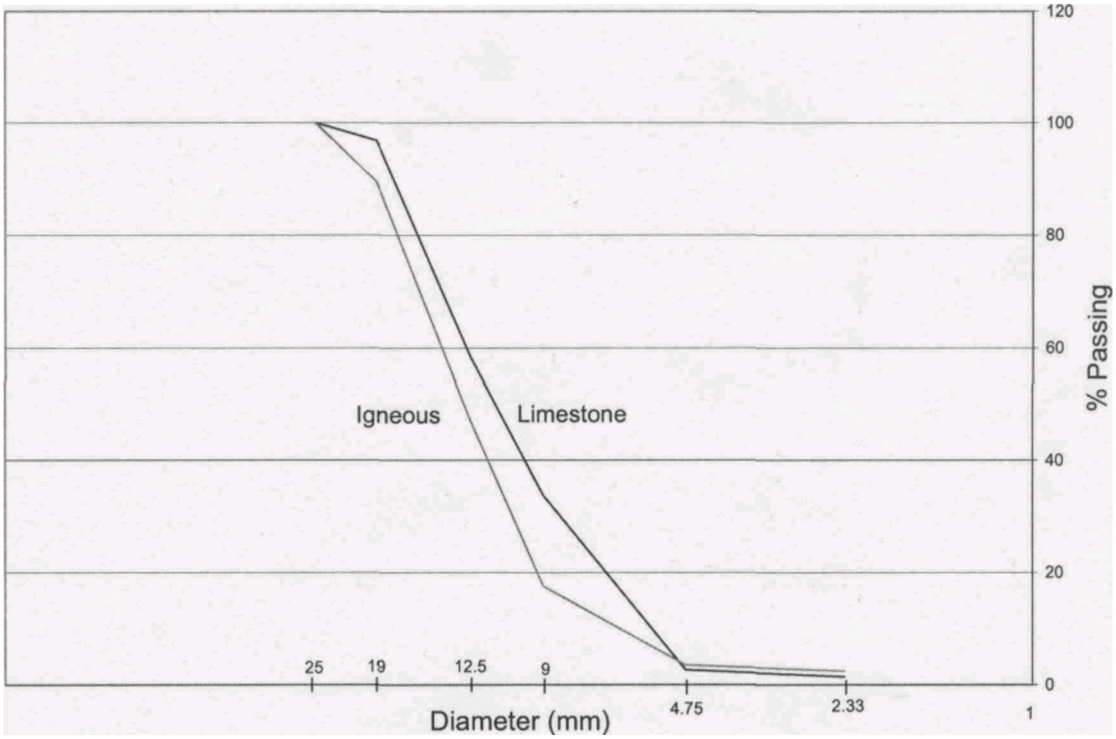

Figure III. 1 Coarse Aggregate Gradations

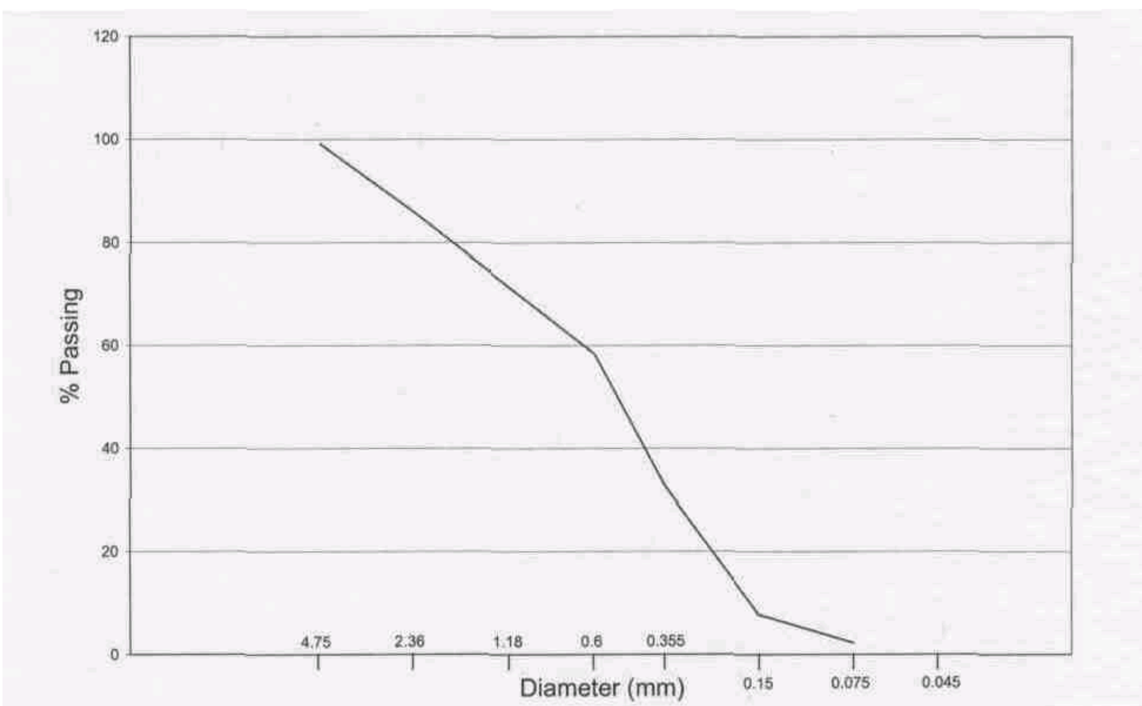

Figure III. 2 Fine Aggregate Gradation

APPENDIX IV – GGBFS and PORTLAND CEMENT COMPOSITIONS

Table IV. 1 GGBFS Chemical Composition

Chemical Compound	Percentage
Silicon Oxide (SiO_2)	37.22
Aluminum Oxide (Al_2O_3)	7.78
Iron Oxide (Fe_2O_3)	1.01
Calcium Oxide (CaO)	37.62
Magnesium Oxide (MgO)	10.98
Sulfur Trioxide (SO_3)	2.53
Sodium Oxide (Na_2O)	0.30
Potassium Oxide (K_2O)	0.33
Titanium Oxide (TiO_2)	0.43
Phosphorus Oxide (P_2O_5)	< 0.01
Manganese Oxide (Mn_2O_3)	0.56
Strontium Oxide (SrO)	0.04
Chromium Oxide (Cr_2O_3)	0.01
Zinc Oxide (ZnO)	< 0.01
Alkalies as Na_2O	0.52
Loss of Ignition	0.56

Table IV. 2 Portland Cement Chemical Composition and Fineness

Chemical Compound	Percentage			
	Cement A	Cement B	Cement C	Cement D
Silicon Oxide (SiO_2)	20.53	19.68	18.95	19.84
Aluminum Oxide (Al_2O_3)	4.98	4.51	5.66	5.07
Iron Oxide (Fe_2O_3)	2.22	3.13	2.44	2.53
Calcium Oxide (CaO)	65.53	62.53	62.58	63.04
Magnesium Oxide (MgO)	1.51	3.68	3.51	2.55
Sulfur Trioxide (SO_3)	3.17	3.33	3.24	2.45
Sodium Oxide (Na_2O)	0.06	0.13	0.24	0.19
Potassium Oxide (K_2O)	0.56	1.36	0.90	0.53
Titanium Oxide (TiO_2)	0.23	0.21	0.03	0.25
Phosphorus Oxide (P_2O_5)	0.06	0.04	0.07	0.12
Manganese Oxide (Mn_2O_3)	0.05	0.05	0.13	0.11
Strontium Oxide (SrO)	0.04	0.06	0.06	0.07
Chromium Oxide (Cr_2O_3)	0.01	0.02	0.01	0.01
Zinc Oxide (ZnO)	< 0.01	0.09	< 0.01	0.02
Alkalies as Na_2O	0.42	1.02	0.83	0.54
C_3S	65	61	60	61
C_2S	10	11	9	11
C_3A	9	7	11	9
C_4AF	7	10	7	8
Loss of Ignition	1.13	1.15	1.88	3.03
Blaine Fineness (m^2/kg)	361	364	322	367
Reported Mill Sheet Fineness (m^2/kg)	348	380	370-380	370-375

Appendix V – EXPERIENCE SURVEY

Several Departments of Transportation and Paving Associations around the Midwest were surveyed on their assumptions, usage, problems, and solutions when dealing with GGBFS in paving concrete. The survey can be seen below followed by each contact's responses.

- Have you been involved in any projects that use GGBFS? If so, how many over the last two years?
- What opinions or assumptions do you have about the use of GGBFS?
- Did those change once you started working with GGBFS?
- What manufacturer and grade GGBFS is commonly used in your projects?
- What cement replacement level is typically used in your projects?
- What type of aggregates are used in your projects?
- What type and/or brand of cements are used in your projects?
- What types of problems, if any, have you encountered with the use of GGBFS?
- Are problems magnified with higher or lower replacement level?
- Type and brand of admixtures used (air entrainment, super plasticizer)
- Do these admixtures significantly affect the performance of your GGBFS concrete?
- Is GGBFS ever combined with other mineral admixes such as silica fume or fly ash?
- Have you experienced any benefits from the combination of multiple mineral additives?
- Time of year project was performed (mainly when was the concrete poured and how was it allowed to cure)
- If more than one curing method, which have you found to be the most beneficial for scaling resistance? Concrete strength?
- Do you impose restrictions on what time of the year GGBFS is used in your projects?

Interview 1 on July 1, 2002 with Minnesota DOT

- Contact has been involved in 5 to 10 GGBFS projects over the past two years.
- The two main assumptions with the use of GGBFS were that it can cause low resistance to scaling, and due to its fineness more water may be needed to coat the particles.
- Grade 120 is most commonly used at a replacement level of 35%.
- One project in western Minnesota combined the use of GGBFS and Fly Ash.
- The GGBFS restrictions are up to the contractor, but use usually ends in October and to keep scaling down GGBFS is not used below 40^o F.

Interview 2 on July 1, 2002 with Illinois DOT

- The contact has been involved in a couple of GGBFS projects. GGBFS is more commonly used in Chicago than through out the state. In 1999, 2000 yd^3 or 131 tons of GGBFS was used.
- Their assumptions included; a slower strength gain in cooler applications (50^o-60^oF), but a more steady strength gain during the summer, and concrete containing GGBFS has lower surface durability.
- The GGBFS goes through a certification process and must be Grade 100 or 120. Typically, a 25% GGBFS replacement level is used.
- For High Performance Concrete used in bridge decks GGBFS is combined with (High Reactivity) HR Metokalin. This results in good permeability properties and lowers the paste level to reduce shrinkage.
- For high early strength patch mixes GGBFS is used with Type III cement and Silica Fume.
- For reconstruction projects a combination of PC, Class F fly ass, silica fume, and GGBFS has been tried. Other blended cements with 25% GGBFS and 20% Class C FA have also been used.
- There is a temperature restriction. GGBFS should not be used when the air temperature is less than 40^oF. If it is, the engineer may change the mix design to compensate.

Interview 3 on August 14, 2002 with Iowa DOT

- Most of major projects use GGBFS, at the very least 50% use it. It has been used consistently over the last 3 to 4 years.
- When they first started using GGBFS, they were worried about scaling. But this was based on a Canadian study done under severe conditions. Iowa's first project with GGBFS was done in the mid-90's and no problems have occurred.
- Not much was known beyond that, but the marketing and information on how to use GGBFS made them comfortable with the material. The increased workability and decreased permeability were two positives for the use of the material.

- Either grade 100 or 120 must be used. The GGBFS is used in blended cements rather than adding the GGBFS separately. The two commonly used blended cements contain either 20% or 35% GGBFS.
- The main problem experienced is the slow set time in colder temperatures.
- Most projects are a ternary mix combining the blended cement with class C fly ash. Bridge decks use GGBFS in HPC concrete. This gives the same desired permeability and strength characteristics of Silica Fume Concrete; it just takes a year to develop them.
- Currently, there are no temperature or seasonal restrictions on using GGBFS, but they may be setting some up.

Interview 4 on August 14, 2002 with Indiana DOT

- Contact has not been directly involved with GGBFS. It is the contractor's choice on whether to use GGBFS or not.
- Indiana has an appropriate materials list that lists which manufactures can be used, and a minimum of Grade 100 GGBFS must be used.
- Typical replacement level is 30 %, and this is replaced at a 1 to 1 substitution by mass.
- Any problems that occur are reported to the contact, and there have not been any GGBFS problems reported.
- Some bridge projects have combined the use of GGBFS and Silica Fume.
- A seasonal restriction is placed on the use of GGBFS. It can be used between April 1st and October 15th.

Interview 5 on October 23, 2002 with Concrete Paving Association of Minnesota

- They have been involved in several GGBFS projects over the past few years.
- They assumed strength gain and set times would be slower. Also assumed was that more water would be needed to coat these fine particles. They observed that more water was needed to coat the particles, but after vibrating the concrete they "got the water back".
- Grade 100 is the most commonly used GGBFS, and is used at a 35% replacement level.
- In Minnesota the max allowed water-cement ratio is .40. There are incentives if .35 is reached and possible penalties if it goes above .40. With GGBFS, it is more difficult to stay below .40, since GGBFS requires more water. The concrete usually comes out of the truck stiff but vibrating it brings out some of the water.
- They do not allow the combination of GGBFS with other mineral admixtures. A few years ago it was combined with fly ash in a project. As a result, the sections of highway had to be removed due to cracking.
- There is a max temperature restriction of 90^{o}F. They try not to impose restrictions, but the contractors need to keep the concrete from freezing and cracking. They are responsible for removing and replacing any damaged concrete. To prevent, this from happening in colder weather, either hot water is used or high early strength cements.

- The Wabasha Bridge in St Paul is a mass concrete project that used 70% GGBFS. High replacement levels keep the temperatures more consistent throughout the concrete.
- In SW Minnesota, quartzite, niess, and expansive sands are used. The MinnDOT specification states that to prevent expansion GGBFS must be used. Recently the spec has changed to allow a choice between GGBFS or a 30% replacement of fly ash.
- In the Twin Cities, a permeability specification must be met for high performance concrete (HPC); usually GGBFS is used to meet this specification.

Summary of Statements

Most users were familiar with the pros and cons of using GGBFS in paving concretes. It is used for its ability to increase concrete strength, but it is known that there are scaling issues. Because GGBFS reacts slowly, it is susceptible to salt scaling and freeze thaw damage. In response to this behavior restrictions on the use of GGBFS have been set. In most cases GGBFS will not be used when the air temperature drops below 40^{o}F. A common seasonal restriction is that GGBFS can only be used between the beginning of April and the middle of October. Another way to aid in scaling and freeze-thaw resistance is to combine the use of GGBFS with Fly Ash. Other mineral admixtures are also being combined with GGBFS for other applications. Both Silica Fume and Metokalin have been combined with GGBFS for high performance concrete used in bridge decks. Both Grade 100 and 120 are used, but never below Grade 100.

APPENDIX VI – AVERAGED COMPRESSIVE STRENGTH

Table VI. 1 Mix Descriptions

Mix Number	% Slag	Cement	Aggregate
2	0	Cement A	Limestone
3	30	Cement A	Limestone
4	50	Cement A	Limestone
5	0	Cement A	Igneous
6	30	Cement A	Igneous
7	50	Cement A	Igneous
8	0	Cement A	Limestone
9	30	Cement A	Limestone
10	50	Cement A	Limestone
11	0	Cement A	Igneous
12	30	Cement A	Igneous
13	50	Cement A	Igneous
14	0	Cement B	Limestone
15	30	Cement B	Limestone
16	50	Cement B	Limestone
17	0	Cement B	Igneous
18	30	Cement B	Igneous
19	50	Cement B	Igneous
20	0	Cement C	Limestone
21	30	Cement C	Limestone
22	50	Cement C	Limestone
23	0	Cement C	Igneous
24	30	Cement C	Igneous
25	50	Cement C	Igneous
26	0	Cement D	Limestone
27	30	Cement D	Limestone
28	50	Cement D	Limestone
29	0	Cement D	Igneous
30	30	Cement D	Igneous
31	50	Cement D	Igneous

Table VI. 2 Air Unadjusted Compression Data (psi)

	Test Day					
Mix #	**3 day**	**7 day**	**14 day**	**28 day**	**56 day**	**365 day**
2	2535	3018	3767	4027	4727	5407
3	1626	2278	2992	3804	3970	4838
4	1136	1856	2756	3322	4206	4728
5	3236	4040	4352	4962	4884	5586
6	1908	2444	2894	3446	3700	4619
7	1436	2214	2878	3556	3814	4404
8	1384	2342	3612	4028	4370	5028
9	784	1616	2762	3477	4118	4682
10	654	1286	1880	2668	3904	4669
11	1094	2312	2830	3788	4102	4573
12	638	1490	2157	2602	3082	4204
13	680	1178	1908	2484	3234	4146
14	3050	3276	4008	4128	4612	5432
15	2384	3014	3654	4274	4632	5640
16	1730	2628	3612	4218	4644	5310
17	2974	3454	3672	4158	4386	5124
18	2310	2806	3278	3924	4196	4840
19	1646	2392	3134	3810	4138	4619
20	3066	3704	4038	4460	4778	5394
21	2702	3500	4424	4902	5386	6150
22	1882	3108	4182	4778	5220	5708
23	3144	3668	3962	4306	4628	5460
24	2544	3108	3682	4244	4682	5208
25	1888	2982	3698	4202	4620	5138
26	3050	3698	4150	4454	4800	5538
27	2488	3386	4022	4640	5014	5730
28	1888	2728	3966	4680	5130	5934
29	3220	3746	4336	4674	4856	5522
30	2426	3004	3582	4150	4654	5236
31	1932	2756	3362	3832	4050	4750

Table VI. 3 Air Adjusted Compression Data (psi)

	Test Day					
Mix #	**3 day**	**7 day**	**14 day**	**28 day**	**56 day**	**365 day**
2	2553	3040	3794	4056	4761	5446
3	1662	2328	3058	3887	4057	4944
4	1169	1911	2837	3420	4330	4867
5	3259	4069	4383	4998	4919	5546
6	1979	2535	3001	3574	3837	4454
7	1436	2214	2878	3556	3814	4404
8	1394	2359	3638	4057	4401	4992
9	769	1607	2714	3441	4111	4728
10	669	1305	1921	2704	3961	4534
11	1056	2232	2732	3657	3961	4540
12	662	1545	2237	2698	3196	4054
13	672	937	1883	2461	3205	4200
14	3050	3276	4008	4128	4612	5432
15	2384	3014	3654	4274	4632	5640
16	1718	2609	3586	4188	4611	5272
17	2995	3479	3698	4188	4418	5161
18	2310	2806	3278	3924	4196	4840
19	1682	2444	3203	3893	4229	4713
20	3088	3731	4067	4492	4812	5433
21	2645	3427	4331	4799	5273	6021
22	1869	3086	4152	4744	5183	5668
23	3144	3668	3962	4306	4628	5460
24	2491	3043	3605	4155	4584	5099
25	1836	2899	3595	4085	4492	4995
26	3050	3698	4150	4454	4800	5538
27	2419	3292	3910	4511	4875	5571
28	1902	2748	3995	4714	5167	5977
29	3152	3667	4245	4576	4754	5406
30	2426	3004	3582	4150	4654	5236
31	1905	2717	3315	3778	3993	4638

APPENDIX VII – DEICER SCALING RESULTS

VII.1 VISUAL RATINGS

Table VII. 1 Visual Ratings Key

Rating	Condition of Surface
0	No Scaling
1	Very light scaling (3mm [1/8in.] depth, max, no coarse aggregate visible)
2	Slight to moderate scaling
3	Moderate scaling (some coarse aggregate visible)
4	Moderate to severe scaling
5	Severe scaling (coarse aggregate visible over entire surface)

Table VII. 2 Deicer Scaling Batch Key

Mix Number	% Slag	Cement	Aggregate
2A	0	Cement A	Limestone
3A	30	Cement A	Limestone
4A	50	Cement A	Limestone
5A	0	Cement A	Igneous
6A	30	Cement A	Igneous
7A	50	Cement A	Igneous
0C	0	Cement A	Limestone
1C	30	Cement C	Limestone
2C	50	Cement C	Limestone
3C	0	Cement C	Limestone
4C	30	Cement C	Limestone
5C	50	Cement C	Limestone
6C*	30	Cement C	Limestone

* Batch 6C was cured for 56 days instead of 28 days.

M0 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.3	0.7	1.0	1.0	1.0	1.0	1.3
Resin	0.0	0.7	0.7	0.7	1.0	1.0	1.7
Air	0.0	0.0	0.0	0.0	0.0	0.3	1.0
Moist	0.0	0.3	1.3	1.0	1.0	1.3	1.7

M1 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	1.0	1.3	1.7	2.0	3.7	3.7	4.3
Resin	1.0	1.0	1.3	2.0	2.3	3.3	3.3
Air	0.3	1.0	1.0	1.3	1.7	2.3	3.0
Moist	1.0	1.0	1.0	1.0	2.0	2.3	3.0

M2 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.3	1.0	2.0	2.3	3.3	3.7	4.3
Resin	1.0	1.7	2.3	2.7	3.3	4.7	4.7
Air	0.3	1.0	1.0	1.0	2.3	3.3	3.7
Moist	2.3	2.3	2.7	2.7	3.7	4.0	4.3

M3 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.7	1.7	1.7	1.3	2.0	2.0	2.3
Resin	0.0	2.0	1.7	1.7	1.7	2.0	2.3
Air	0.7	1.7	1.3	1.3	1.3	1.7	1.3
Moist	0.7	1.7	1.7	1.3	1.3	1.7	1.7

M4 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.3	2.7	2.0	2.7	2.0	2.7	3.0
Resin	0.3	2.3	1.7	2.0	2.3	2.3	3.0
Air	0.0	1.7	1.0	1.7	1.7	2.0	2.3
Moist	0.3	2.3	1.7	2.3	2.3	3.0	3.3

M5 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.3	1.3	2.3	2.3	3.3	4.3	5.0
Resin	0.3	0.7	1.7	2.0	2.7	3.7	4.3
Air	0.3	1.0	1.7	1.3	2.3	3.7	3.3
Moist	2.0	2.7	2.3	2.3	3.3	4.3	3.7

M6 Cement C, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.7	1.0	1.3	2.3	2.7	3.0	3.0
Resin	1.0	1.3	1.3	2.0	2.3	3.0	3.3
Air	0.0	1.3	1.3	1.3	2.0	2.3	2.7
Moist	0.7	0.7	1.0	1.0	1.0	1.3	2.7

M2 Cement A, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	0.0	2.0	3.3	4.0	5.0	5.0	5.0
Resin	0.0	1.7	3.0	3.0	4.0	4.0	4.0
Air	3.0	3.0	3.7	3.7	3.7	3.7	3.7
Moist	0.7	1.0	1.3	1.7	2.3	2.3	2.3

M3 Cement A, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	3.3	4.3	5.0	5.0	5.0	5.0	5.0
Resin	3.0	4.0	5.0	5.0	5.0	5.0	5.0
Air	4.0	4.0	4.0	4.3	4.3	4.3	4.3
Moist	4.7	5.0	5.0	5.0	5.0	5.0	5.0

M4 Cement A, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	4.3	4.7	5.0	5.0	5.0	5.0	5.0
Resin	4.7	5.0	5.0	5.0	5.0	5.0	5.0
Air	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Moist	5.0	5.0	5.0	5.0	5.0	5.0	5.0

M5 Cement A, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	2.7	3.0	3.3	3.7	4.7	4.7	4.7
Resin	2.0	3.0	3.0	3.0	3.0	3.0	3.0
Air	1.0	3.0	3.0	3.0	3.0	3.0	3.0
Moist	2.3	3.0	3.0	3.0	3.0	3.0	3.0

M6 Cement A, 3 Block Average							
	# Cycles						
Curing Method	5	10	15	25	50	75	100
Wax	2.7	3.0	3.0	4.0	4.3	4.3	4.7
Resin	2.7	3.0	3.0	3.0	3.7	3.7	4.0
Air	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Moist	3.0	3.3	3.7	3.7	3.7	3.7	4.0

M7 Cement A, 3 Block Average							
	# Cycles						
Curing Method	**5**	**10**	**15**	**25**	**50**	**75**	**100**
Wax	3.0	4.0	4.3	4.3	4.3	5.0	5.0
Resin	3.3	4.3	4.3	4.3	4.3	5.0	5.0
Air	3.0	3.0	3.0	3.0	3.0	3.0	4.0
Moist	4.7	5.0	5.0	5.0	5.0	5.0	5.0

VII.3 WASH OFF DATA

3 Block Average Washoff - M2 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	6.9	6.4	46.2	6.8
10	38.8	22.4	80.2	9.6
15	98.4	50.7	127.2	11.0
20	126.1	61.5	147.2	11.4
25	176.4	80.9	163.3	12.0
30	217.5	129.0	177.5	12.7
35	234.8	141.9	185.7	13.1
40	249.0	152.8	191.7	13.4
45	264.5	172.7	196.4	13.8
50	274.4	180.5	202.7	14.3
55	297.0	198.4	209.7	16.9
60	309.3	208.4	215.3	17.5
65	333.1	225.4	231.5	19.0
70	349.1	244.5	249.6	19.5
75	360.1	256.7	261.2	19.8
80	365.2	263.9	269.2	21.2
85	371.8	274.6	278.8	23.3
90	379.2	295.3	284.4	27.6
95	380.9	298.2	287.7	28.9
100	382.7	303.4	289.7	29.4

3 Block Average Washoff - M3 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	150.9	114.0	158.3	929.0
10	420.3	307.1	239.3	1266.5
15	597.2	430.2	314.0	1463.5
20	668.0	476.3	347.6	1499.7
25	735.5	499.1	459.7	1523.2
30	753.1	506.9	502.1	1539.0
35	773.5	520.1	533.3	1587.6
40	783.0	531.0	566.5	1591.7
45	794.7	543.3	592.7	1618.2
50	802.4	551.9	613.5	1638.2
55	850.6	614.1	654.3	1769.2
60	887.8	652.6	687.0	1859.2
65	904.2	666.0	711.6	1866.1
70	914.2	675.0	737.5	1877.3
75	923.6	678.4	745.0	1883.0
80	935.0	706.9	786.4	1891.4
85	938.8	710.8	795.7	1896.8
90	941.2	717.2	798.4	1897.7
95	943.6	720.6	806.4	1900.1
100	945.2	724.1	808.9	1903.5

3 Block Average Washoff - M4 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	962.6	775.6	628.0	1969.6
10	1267.7	1230.1	833.2	2186.0
15	1686.6	1635.9	1085.3	2296.9
20	1834.4	1740.3	1222.8	2326.2
25	1904.8	1785.9	1328.8	2354.5
30	1940.8	1840.2	1381.5	2378.8
35	1953.8	1850.6	1483.6	2383.6
40	1975.6	1868.3	1533.8	2411.0
45	1991.7	1884.1	1540.7	2414.8
50	2021.9	1905.4	1577.2	2604.0
55	2052.4	1934.4	1596.8	2657.4
60	2059.6	1939.8	1614.3	2678.5
65	2064.1	1950.3	1668.5	2701.1
70	2070.7	1959.5	1682.9	2709.3
75	2084.0	2016.1	1686.9	2714.2
80	2088.3	2025.5	1688.2	2715.8
85	2089.9	2029.1	1698.6	2719.0
90	2092.3	2033.5	1713.2	2722.0
95	2093.4	2035.0	1731.4	2724.2
100	2096.1	2038.8	1746.2	2730.6

3 Block Average Washoff - M5 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	42.7	36.2	32.5	21.2
10	189.1	91.0	62.4	32.0
15	372.8	127.2	79.8	37.6
20	561.8	173.4	98.8	42.8
25	730.5	254.0	119.1	46.7
30	881.7	326.8	138.5	51.1
35	936.9	367.7	145.6	53.2
40	985.7	412.0	153.7	55.3
45	1062.9	461.4	167.1	59.8
50	1100.8	479.6	169.0	60.6
55	1127.8	496.2	173.3	62.0
60	1151.1	512.4	176.9	63.9
65	1161.8	525.4	180.4	65.6
70	1169.4	528.7	181.1	66.3
75	1181.6	537.0	183.1	67.9
80	1190.3	544.0	184.9	69.9
85	1199.0	548.1	189.0	72.5
90	1209.7	551.5	192.4	74.6
95	1218.7	556.4	195.5	75.7
100	1225.1	557.9	196.8	76.4

3 Block Average Washoff - M6 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	Wax	Resin	Air	Moist
0	0.0	0.0	0.0	0.0
5	71.4	71.2	90.6	197.9
10	320.1	271.2	145.7	372.8
15	517.0	469.0	183.0	445.3
20	739.1	718.7	221.5	498.0
25	904.3	903.1	249.3	512.9
30	1062.7	1098.2	283.9	540.6
35	1136.6	1173.2	295.2	547.5
40	1203.5	1257.7	311.4	553.3
45	1274.9	1343.3	330.1	575.6
50	1297.7	1379.3	336.2	578.4
55	1321.0	1417.7	348.2	583.7
60	1339.0	1441.2	354.2	585.2
65	1350.6	1453.5	357.4	586.2
70	1366.1	1468.8	363.5	587.0
75	1375.2	1478.3	370.3	592.2
80	1386.3	1495.3	375.6	597.8
85	1402.9	1506.7	383.6	601.6
90	1418.7	1524.2	389.7	628.2
95	1428.0	1533.1	393.5	637.2
100	1439.0	1549.3	401.3	640.3

3 Block Average Washoff - M7 Cement A				
	Washoff Normalized to Area (g/m^2)			
Cycle	Wax	Resin	Air	Moist
0	0.0	0.0	0.0	0.0
5	235.0	726.2	190.3	731.1
10	465.4	1040.5	275.0	1043.0
15	612.0	1304.4	349.3	1196.0
20	764.0	1588.2	432.9	1311.8
25	799.2	1630.9	449.9	1333.3
30	873.7	1728.7	484.5	1374.6
35	930.0	1812.4	510.3	1401.7
40	990.9	1905.2	530.7	1421.0
45	1016.5	1954.0	546.0	1428.7
50	1059.5	2014.5	566.4	1443.5
55	1081.7	2057.5	575.5	1449.3
60	1100.5	2102.5	586.1	1457.1
65	1113.2	2129.2	591.6	1459.0
70	1136.3	2191.8	604.0	1467.7
75	1169.0	2214.3	620.2	1470.6
80	1188.6	2255.4	646.8	1478.7
85	1212.3	2305.6	679.2	1488.6
90	1224.6	2371.1	697.5	1495.0
95	1230.7	2404.4	720.5	1502.2
100	1240.1	2427.9	752.7	1512.3

3 Block Average Washoff - M0 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	28.6	7.2	1.8	3.1
10	61.7	16.3	2.6	18.1
15	77.8	18.8	3.2	19.4
20	89.2	19.2	5.1	22.1
25	96.3	28.4	6.9	25.1
30	105.0	31.0	7.3	26.2
35	107.2	32.2	9.1	27.3
40	110.4	33.1	9.8	28.0
45	114.0	43.3	16.8	28.0
50	115.6	45.1	18.3	28.0
55	117.3	48.3	18.6	28.0
60	118.4	49.0	19.7	28.8
65	127.3	49.7	20.3	30.5
70	129.1	49.7	26.1	30.9
75	129.6	51.8	29.3	30.9
80	131.0	52.7	31.0	31.8
85	132.6	53.3	31.6	31.8
90	133.6	53.3	31.6	31.8
95	134.4	60.1	38.2	32.2
100	134.4	60.1	38.2	32.2

3 Block Average Washoff - M1 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	24.0	14.1	10.4	47.6
10	61.7	31.7	67.9	88.8
15	191.5	111.0	111.2	116.7
20	285.1	173.3	144.9	128.4
25	345.3	205.8	180.7	134.5
30	369.0	211.3	193.1	138.1
35	504.7	235.9	205.5	149.2
40	592.5	244.4	209.1	151.1
45	615.5	256.8	207.9	152.7
50	638.8	266.4	222.0	156.3
55	659.6	283.2	230.4	175.4
60	675.0	296.7	237.1	176.3
65	682.2	302.5	263.6	179.3
70	685.3	303.9	268.5	180.3
75	690.2	307.0	297.1	184.4
80	691.8	309.7	326.9	187.9
85	699.5	313.8	329.7	190.4
90	701.5	315.2	338.5	190.4
95	703.7	317.9	342.7	192.2
100	705.3	319.4	344.0	193.4

3 Block Average Washoff - M2 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	19.8	133.3	50.5	331.4
10	147.2	338.6	105.2	455.8
15	337.4	547.8	175.1	516.5
20	494.4	595.7	185.3	527.3
25	577.5	611.6	190.1	533.2
30	683.2	700.4	226.1	570.5
35	745.2	720.8	302.9	662.7
40	772.6	733.9	331.0	686.5
45	789.3	759.3	345.2	711.7
50	819.0	788.6	354.5	734.8
55	806.7	807.9	359.9	746.4
60	819.5	829.1	366.2	765.2
65	823.4	839.4	368.6	772.9
70	833.1	860.7	421.4	810.8
75	845.6	884.9	453.5	836.6
80	858.4	907.1	479.8	868.5
85	871.2	920.9	487.9	873.9
90	877.1	927.1	490.9	878.7
95	877.8	928.3	558.8	879.3
100	886.9	934.0	560.8	881.8

3 Block Average Washoff - M3 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	24.5	18.7	23.6	50.5
10	52.0	55.4	41.1	70.5
15	87.7	78.3	50.7	83.5
20	99.3	92.4	61.0	87.8
25	113.3	100.3	83.0	93.4
30	119.0	113.2	88.1	102.3
35	127.4	117.8	95.1	106.0
40	133.1	128.6	98.8	129.6
45	138.3	132.6	101.5	135.6
50	141.8	133.8	105.0	135.9
55	143.5	135.6	105.8	136.7
60	147.1	143.2	107.6	152.2
65	149.0	146.8	111.7	152.6
70	152.4	150.6	114.1	153.3
75	153.0	154.7	115.5	154.8
80	153.7	156.4	116.8	155.5
85	153.7	157.5	117.4	156.0
90	153.7	157.8	119.3	157.0
95	155.0	157.8	119.7	157.7
100	155.0	164.7	119.7	157.7

3 Block Average Washoff - M4 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	29.2	18.8	13.9	31.1
10	99.6	77.1	32.9	107.2
15	149.7	90.7	69.9	140.4
20	212.5	109.4	89.8	156.2
25	238.4	129.7	98.8	187.0
30	249.4	150.6	109.9	191.1
35	263.5	174.2	147.1	244.0
40	272.2	198.0	152.7	252.8
45	281.2	216.0	157.1	257.5
50	292.2	235.1	167.5	287.7
55	298.5	285.5	172.8	324.1
60	302.9	295.7	178.6	336.9
65	307.8	307.1	183.9	343.1
70	324.7	330.7	190.6	352.6
75	331.9	343.6	200.9	364.0
80	340.0	352.2	215.4	369.9
85	344.7	365.7	222.4	377.0
90	347.2	367.2	223.6	378.3
95	361.8	386.1	231.5	386.8
100	377.4	389.2	234.0	397.0

3 Block Average Washoff - M5 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	19.4	19.3	24.1	73.8
10	55.6	36.8	52.8	263.5
15	453.1	225.3	86.3	569.6
20	611.4	322.7	139.2	607.8
25	658.1	364.9	166.0	629.9
30	701.4	395.1	225.6	633.9
35	729.4	414.2	269.1	639.1
40	783.2	423.2	299.6	643.0
45	862.0	454.5	342.4	673.4
50	918.8	481.4	372.6	688.5
55	940.9	505.4	405.8	702.3
60	969.6	528.1	448.8	715.9
65	985.1	539.2	473.1	718.8
70	998.1	549.4	483.3	724.4
75	1002.1	551.7	485.8	724.9
80	1009.9	553.6	499.9	733.2
85	1020.8	559.6	503.3	737.1
90	1024.5	561.6	504.0	739.1
95	1025.7	564.6	511.1	744.7
100	1028.1	567.2	517.9	740.4

3 Block Average Washoff - M6 Cement C				
	Washoff Normalized to Area (g/m^2)			
Cycle	**Wax**	**Resin**	**Air**	**Moist**
0	0.0	0.0	0.0	0.0
5	35.5	43.8	15.9	9.1
10	94.5	87.1	52.0	29.2
15	172.9	117.0	109.7	42.3
20	244.7	175.9	147.2	59.4
25	290.9	208.2	169.5	63.4
30	321.1	227.3	174.3	66.5
35	330.4	236.0	187.9	78.8
40	337.4	243.8	195.1	84.8
45	342.3	251.5	202.1	90.0
50	344.4	259.4	206.3	91.5
55	347.4	275.4	210.7	100.1
60	347.8	278.3	212.6	109.8
65	352.3	281.1	220.4	114.3
70	353.3	283.3	222.4	116.2
75	356.3	285.0	223.6	118.4
80	358.4	286.4	225.0	128.7
85	377.1	299.1	229.3	153.4
90	385.1	300.8	231.2	164.1
95	388.5	304.2	233.1	192.6
100	390.4	306.4	235.3	212.9